ADDIS ABABA SCIENCE AND TECHNOLOGY UNIVERSITY COLLEGE OF ARCHITECTURE AND CIVIL ENGINEERING

EVALUATIONS OF EMBANKMENT DAM STABILITY ANALYSIS: THE CASE OF ARJO DHIDHESSA EMBANKMENT DAM

A Thesis Submitted to College of Architecture and Civil Engineering of Addis Ababa Science and Technology University

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Abstract

Seepage and slope stability analysis is a vital task during embankment dam design. The consequence of dam failure is one of the major hazards to human life as well as to infrastructure. Studies about a natural dam failure are very common, but the thesis included numerical modeling and detailed analysis of dam conditions. The Arjo Dhidhessa embankment dam was designed for irrigation purpose. Seepage was not analyzed during the embankment dam design level and hence this study attempted to analyze the stability of the constructed coffer dam.

The primary objective of the thesis is to evaluate the seepage and stability of the Arjo Dhidhessa embankment dam by steady-state and transient analysis types.

Stability and seepage has been analyzed using the popular geotechnical software called Geo-Slope. Slope stability analyzed by Morgenstern-price method under limit equilibrium.

The slope is potentially stable throughout the steady-state and transient analysis and the total flux through the dam is decreasing with increasing time that shows the dam has no significant seepage effect.

The height of the cofferdam increased in 6m during overtopping occurred to mitigate flood without as per design and abutments and foundation needed grouting.

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List of Acronyms and Abbreviations

ADS-1	saddle dam axis one
α	*base* angle
BCM	billion meter cubes
C′	effective Cohesion
cu	undrained cohesion
D/S	Down Stream
E	interslice normal force
Ei	left interslice normal force
Ei+1	right interslice normal force

EL	elevation
Ff	horizontal force equilibrium
Fm	moment equilibrium
FOS	factor of safety
FRL	full reservoir level
Ha	Hectare
H	Height of Dam
h	Horihontal
Hd	Head over Crest
ICOLD	International Commission on Large Dams
IS	Indian Standard
L	Length of base of slice
λ	is the percentage function used
MWL	Maximum Water Level
N	slice base normal force
NE	Northeast
OWWDSE	Oromiya water works Design and supervision enterprise
φ	angle of Internal Friction
φ′	effective angle of friction
Q	discharge
RCC	Reinforced compacted concrete
S	slope
Σ	summation
σ	total stress
τf	shear strength at failure
θ	interslice force inclination angle
u	pore water pressure
U/S	upstream
USSD	United States Society on Dams
USBR	united States Bureau of Reclamation

V	velocity of Flow
W	slice weight
W	top Width of Crest of Dam
WWDSE	Water Works Design & Supervision Enterprise
X	interslice shear force
Xi	left interslice shear force
Xi+1	right interslice shear force
Z	pore pressure depth

1 Introduction

1.1 Background

The design and construction of an embankment dam is one of the key challenging in the field of Geotechnical engineering, because of the nature of the varying foundation condition and the range of properties of the material available for construction (U.S. Army corps engineers 2004). The major advantages of the embankment dams are easily adapting to the foundation and accommodate even in difficult site condition (Jansen et al. 1988).

Failures of the dam can consequences loss of life & properties if not properly designed, constructed and operational. It may result from overtopping, sudden drawdown, excessive seepage, instability of the foundation, piping through & under the dam. The design of earth and rock-fill dams involves in many considerations that must be examined before initiating detailed stability analysis. Geological and subsurface exploration, embankment dam materials available for construction must be carefully studied. Successful designing and construction of an embankment dam should be fulfill the following technical and administrative requirements (U.S. Army corps engineers 2004).

A) Technical requirements:

I. Dam foundation and abutment must be stable at all static and dynamic loading conditions.
II. Should have a special design for control and collect seepage though the foundation, abutments and embankment.
III. The outlet capacity of the spillway must be sufficient to prevent overtopping of embankment by reservoir.

B) Administrative requirements:

I. Environmental responsibility
II. Detailed operation and maintenance methods
III. Monitoring plans
IV. Adequate instrumentation

V. Records for all operation and maintenance activities

VI. Emergency action plan (included identification and immediate response). U.S. Bureau of Reclamation, 1991.

The Arjo-Dhedhessa irrigation project envisages construction of an earth and rock fill dam. Arjo-Dhedhessa irrigation project comprises of 47 m high earth fill dam on River Dhidhessa for impounding inflows of the river during the monsoon period. The stored water will be diverted into canal system on the right bank for providing water through a network of canal system for irrigation in the command of 13325ha irrigable lands (OWWDSE. 2009).

1.2 Problem of the Statement

Arjo Dhidhessa embankment dam Design was done by Oromiya Water Works Design and Supervision Enterprise in association with Intercontinental Consultants and Technocrats Pvt. Ltd. (Indian.) During the embankment dam stability analyzed, the seepage analysis was missed both cofferdam and main dam. The transient analysis could be done based on the steady-state analysis as parental analysis. But, the slope stability analysis done was not transferred from steady-state analysis to the transient analysis. However, the construction of the embankment dam has been started by Oromiya Water Works and Construction Enterprise without design document reviewed. Arjo Dhidhessa Cofferdam is an integral part of main dam and constructed in 2015. After construction of the cofferdam was completed 6m of its height increased during overtopping occurred to mitigate the flood without as per design. The construction of main embankment dam was proposed for 2015. Unfortunately, the excavated core of main dam for impervious material is filled with excessive seepage faced the contractor (Oromiya Water Works and Construction Enterprise) within a year before starting the construction of the main dam. The problem of why the cofferdam seepage failure occurred has not been identified. Thus this thesis attempts to study the safety of the cofferdam with respect to maximum seepage occurrence as well as the main dam body.

a) Seepage problem b) in overtopping case 6m height of dam added

c) Diversion channel after overtopping to Wama River
Figure1. 1: Arjo Dhidhessa cofferdam seepage and overtopping problem

1.3 Objective

The general objective of this study is to evaluate Arjo Dhidhessa Embankment dam stability analysis.

***1.4Specific* Objectives**

The specific objectives are:

To identify seepage effect on the cofferdam and main dam

Determination of slope marginal safety for cofferdam and main dam

To evaluate effect of drawdown water table on slope stability and seepage on cofferdam and main dam

2 Literature Review

2.1 Embankment Dams

Embankment dams are made of natural materials excavated or obtained in the surrounding area without any binding. There are two main types of embankment dams: Earth fill and rock fill dams depending on the materials used on the embankment.

An embankment dam can be characterized as an earth fill dam if compacted soils account for over 50% of the placed volume of material. An earth fill dam is constructed primarily of selected engineering soils compacted uniformly and intensively in relatively thin layers and at a controlled moisture content. They usually, consists of impermeable core made of clayey soils, filters and drains usually made of sandy and gravelly soil to prevent the core from being washed out. (Novak et al, 2007).

2.1.1 Homogenous Type of Dam

Homogeneous type of embankment dams is composed, essentially, of the same material throughout the embankment. The material used for construction of this type of dam must be sufficiently impervious to give an adequate water barrier. However, these dams are only for low to moderate height as they have very low slopes. Further, it is very rare that sufficient quantity of homogenous materials would be available within the economic distance from the proposed dam site (Nigatu, 2006)

2.1.2 Zoned Type of Dam

A zoned type earth dam is composed of more than one type of naturally available material. This is the most common type of a rolled fill dam in which zones of materials of considerably more pervious material forms the outer shell and a relatively impervious material forms the central core. The pervious zones may consist of sand, gravel cobbles or rock materials, while the core consists of an impervious soil such as clay, silt or clay gravel mixture. This type of dam is selected when the impervious material is not available in the sufficient quantity near to the Dam site (Nigatu, 2006)

2.1.3 Rock Fills Type of Dam

The rock fill dam consist of three basic elements; (i) a loose rock fill dump, which constitutes the bulk of the dam and resist the thrust of the reservoir, (ii) Impervious facing of the upstream slope with concrete, timber, steel and (iii) Rubble masonry between (i) and (ii) to act as a cushion for the membrane and resist destructive deflections (Nigatu, 2006)

2.2 Design Criteria

An embankment dam must be able to safely with stand static and dynamic loads that may be imposed upon it during its life. If you are an owner or otherwise have responsibility for an embankment dam, inherent in that responsibility is your obligation to ensure the static and dynamic stability of the dam. This unit provides an overview for evaluating embankment dam stability, include:

- Safety evaluation requirements.
- Effect of seepage on embankment dam stability.
- Embankment behavior.

Embankment dams have been built since early times. The general philosophy in design of these dams has been to utilize locally available geologic materials. Design practices have evolved with improved understanding of soil behavior. Construction techniques have evolved with advances in earthmoving and compaction equipment (U.S. Bureau of Reclamation, 1991)

Embankment dams are a preferred choice for sites with wide valleys and difficult foundation conditions because of their flexibility. However, soil is engineering material because of diverse composition, and incomplete understanding of its behavior under all of the stress and boundary condition usually encountered in the field. Soil behavior under load is, in general, highly nonlinear, time dependent, and strain softening. The geologic past, of dam site significantly affects the in service performance of the dam, but these is information is generally not completely known (U.S. Bureau of Reclamation, 1991)

2.2.1 Evaluating Embankment Dam Stability

Soil mechanics, as an engineering science, is a relatively young discipline in engineering education and practices. Earthquake engineering of embankment dams is even younger and somewhat in its formative stages. Although a great deal has been learned and put to use in design and construction of newer dams, there exist in the field a large number of dams designed and built without the benefits of modern understanding of soil behavior and improved construction techniques. When combined, these factors make the stability evaluation of existing embankment dams a difficult and challenging engineering undertaking. Because of uncertainties in problem definition, and an incomplete understanding of soil behavior under all loading conditions encountered, the stability evaluation of existing embankment dams must proceed on a conservative basis (U.S. Bureau of Reclamation, 1991)

Engineers responsible for remedial action usually do not have the full range of options deal with potential responsible that were available at the time the original project is conceived. The must cope with existing conditions, including the presence of the dam itself. Being denied direct access to the foundation under the dam and its appurtenances for inspection and remedial treatment, the engineers must sometimes devise imaginative ways to circumvent the handicap. Often, economics rule against or limit the time available lowering the reservoir water level to facilitate work on the upstream parts of the dam, the reservoir floor, or on the abutments below the normal water surface. For these reasons, remedial work may be more difficult and more expensive than corresponding categories of work would have been at the outset of the project. The structural safety of an embankment dam is dependent primarily on the absence of excessive deformations and pore fluid pressure build-up under all conditions of environment and operation, the ability to safely pass flood flows, and the control of seepage to prevent migration of materials and thus preclude adverse effects on stability (U.S. Bureau of Reclamation, 1991)

2.2.2 Evaluation Requirements

All embankment dam in service, regardless of their age, should be systematically evaluated for their safe performance under all operational conditions. The principal requirement for dam safety evaluation is to protect public safety, life, and property. Hence, all dams must function safely under routine everyday operation as well as under usual conditions such as floods and earthquakes. The potential for adverse incidents, such as excessive seepage, instability, and major damage during floods and earthquakes, needs to be assessed to ensure that the safety of people and property will not be endangered by the dam. If a risk does exist, corrective actions need to be taken (U.S. Bureau of Reclamation, 1991)

2.3 Effect of Seepage on Embankment Dam Stability

All embankment dams are subject to some seepage passing through, under, and around them. If uncontrolled, seepage may be detrimental to the stability of the structure as a result of excessive pore pressure, or by internal erosion. For existing embankment dams, all seepage records compiled during the existence of the structure should be reviewed for significant trends or abnormal changes. The cause of any abnormality should be determined as accurately as possible. Any record any evidence that seepage flows have removed any significant amount of fine-grained soil must be evaluated through field investigations. Turbid flow issuing from a dam or its foundation may be an indication of internal erosion. Seepage should be effectively controlled to preclude structural damage or interference with normal operations. There are several instances in which potential instability problems were identified during routine safety evaluations and corrective actions taken to avert possible dam incidents from happening (U.S. Bureau of Reclamation, 1991).

Table2. 1: Historical record of embankment dam failures and accidents to 1979 for dams of heights 50 feet or greater

CAUSE	FAILURE	INCIDENT
Overtopping	18	7
Flow erosion	14	17
Slope protection damage	-	13
Embankment leakage, piping	23	14
Foundation leakage, piping	11	43
Sliding	5	28
Deformation	3	29
Deterioration	2	3
Earthquake instability	-	3
Faulty construction	-	3
Gate failure	1	3
TOTAL	**77**	**163**

From: development of dam engineering in the united states by E.B.Kollgaard and W.L. Chadwick, (Eds.) pergamon press, New York, 1988.

Seepage failure is the dangerous action to embankment dam instability. From the above evidence 62% of failure and 45% incident caused due to seepage problem.

As data 111 failures shows three main reasons for embankment dam failure

- overtopping at high flood discharge (about 30% of the total failures)
- internal erosion and seepage problems in the embankment (about 20%) and

- internal erosion and seepage problems in the foundation (about 15%) (ICOLD 1995)

Here is we see easily that seepage is a powerful cause for the embankment dam instability.

2.4 Permeability

Permeability is the most important properties that Hydrologists, Geotechnical engineers and ground water hydrology professionals always deal with (Cedergren 1989). Naturally all the soil materials are permeable, means water can flow through the soil by the interconnected pore spaces in the soil. The quantity of permeability is always denoted by the term coefficient of permeability (k). A permeable material must have the ability to be penetrated by another material such as gas or liquid. Most of the soil and rocks with cracks and joints are some common permeable materials which deal with geotechnical works.

The soils are categorised as permeable, semi permeable or impermeable as per the following limits:

Impermeable: with permeability less than 1 x 10^{-6} cm/sec

Semi permeable: with permeability 1x 10^{-6} to 1x 10^{-4} cm/sec.

Permeable: With permeability more than 1x 10^{-4} cm/sec.

The dam embankments should be impermeable. The permeability of the downstream section of embankment should not be less than that upstream. (Department Of Water Resources, 2016)

The USBR measured hydraulic conductivities of well-graded sand and gravel mixtures in the range of about 1 to 5 X 10^{-2} cm/s. (USBR, 1990)

Clean gravels have high hydraulic conductivities, ranging on the order of 1 to 100 cm/s. (USSD, 2011)

2.5 Modes of Failure of Embankment Dams

The most common modes of failure of embankment dams can be separated in three main categories: hydraulic, seepage and structural failures. Some of these mechanisms are depicted in figure2

Hydraulic Failures:

A) Overtopping: when the freeboard of the dam or the capacity of the spillway is insufficient, the flood water will pass beyond the crest of the dam and cause erosion of the crest and the downstream side (figure 2a). Antonia, 2013

B) Erosion of the downstream toe: This is due to heavy cross current from the spillway or tail water. (Antonia, 2013)

C) Erosion of upstream face: This mode of failure is caused by waves on the surface of the reservoir (figure 2b). Antonia, 2013

D) Erosion of the downstream face: This failure is caused by weathering of the face due to heavy rain or due to animals and plants (figure 2c). Antonia, 2013

Seepage Failures:

A) Piping through dam body: During seepage small channels can be formed which transport material downstream and gradually increase (figure 2d). Antonia, 2013

B) Piping through foundation: if in the dam foundation there are highly permeable cavities, fissures, concentrated seepage at a high rate occurs. This leads to erosion and flow of water and soil in the foundation (figure 2e).

C) Sloughing of the downstream side of dam: The downstream toe of the dam becomes saturated and starts eroding causing small slump or slide of the dam which can gradually progress and lead to failure.

Structural Failures:

A) Slide in embankment: If either of the slopes of the embankment is to steep it can be slide. For the u/s slope this is usually triggered by a sudden drawdown (figure 2f).

B) Foundation slide: This mode of failure occurs if the foundation is composed by soft soil and can lead to the whole dam sliding due to water thrust (figure 2g).

C) Earth quake failure: Earthquake loading can lead to failure of the dam itself but also of the foundation and the appurtenant structures. (Antonia, 2013)

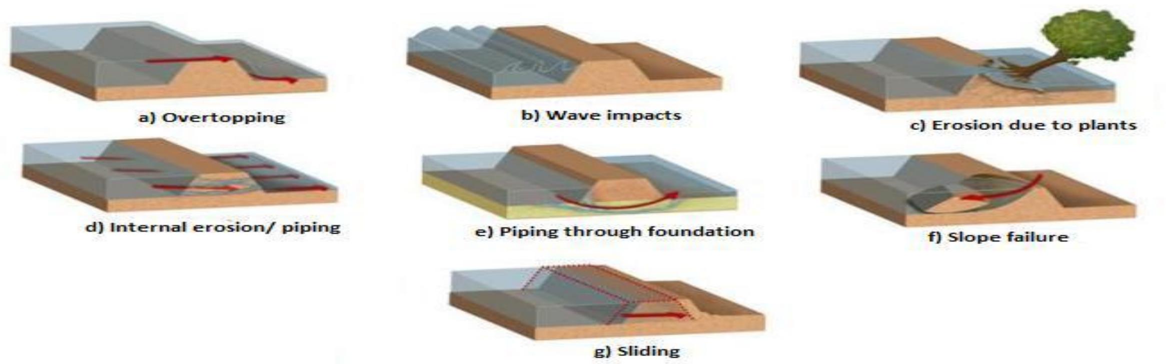


Figure2. 1: Modes of failure of embankment dams (by Zila Deresky, NSF) as cited in Antonia, 2013

2.6 Seepage

Seepage analysis plays an important role in embankment dam engineering. It is required in different scenarios such as in volume change prediction, ground water contamination control, monitoring of leakage amount, slope stability analysis and design of earth structures. The flow net solution for simple unconfined flow cases without boundary conditions for linear partial differential equation has three basic assumptions (Boston, 1937 as cited in Abebe, 2014)

Seepage in embankment dams occurs through the foundation, embankment body or through both parts. Although applying a correct set of boundary conditions is vital in obtaining reliable results from seepage analyses in heterogeneous embankment dams, such boundary conditions are usually applied using certain simplifications. Simplifications of boundary conditions are applied in different ways at the upstream shell, the core, the filters, and the downstream shell. A simple, but a common approach in the finite element or finite difference seepage analysis is to eliminate the shells and the filters because of their high perm abilities. In this approach, only the core is modeled and the reservoir elevations along with the known downstream pore pressures are applied as the upstream and downstream boundary conditions, respectively .(G.Rakhshandehroo and A. Bagherieh, 2006 as cited in Abebe, 2014).

2.6.1 Seepage Controls for Embankment

The importance of the seepage control through embankment is to control the water loss within the acceptable limits and to ensure the safety of the Dam. Water seepage under pressure through soil voids is accompanied by a mechanical drag on the soil particle when these force exceeds, the soil grain movement may take place. A large percentage of the earth dam failures reported by the Sherard & colleagues was due to seepage (Bharat Singh & R.S Varshney ,1995 as cited in Fikadu, 2006). As a result, it is important to control the migration of soil particles resulting piping failure and embankment failure by saturation or seepage forces. The migration is mainly caused due to the lack of filter protection, poor compaction, in proper placement of pervious material in the embankment section and leaching of dispersive soils. The saturation of seepage forces is mainly due to the excessive pore pressure causing slope failure, liquefaction failures due to earthquake shocks, foundation blow out due to excessive uplift and sloughing of downstream toe due to saturation. Therefore, drainage of an embankment is necessary to provide a safe passage to the water, which has entered into the dam body, without developing excessive pore pressure. Two approaches are followed to control the seepage through an embankment dam. The first approach is preventive whereas, the other approach is curative. In earth dam design practice both the approaches are followed in combination. In preventive approach efforts are made in keeping the water out in so far as possible while in the curative approach a safe outlet is provided to water, which has entered in spite of the preventive measures. (Fikadu, 2006)

2.6.2 Current Practices in Seepage Control

There are currently three basic methods for controlling seepage. They are:

- Using filters to prevent soil particle movement caused by seepage.
- Employing methods to reduce the quantity of seepage.
- Using drainage methods to relieve seepage pressures and to collect seepage and convey it to a safe outlet.

Frequently, these methods are used in combination. Effective control of seepage requires that both the dam and its foundation be considered together. Despite the advances that have been made in the design of dams and seepage control methods, significant failures still occur. Seepage was the cause of failure of several modern dams. Each failure has brought new understanding and advances in the control of seepage. In other words, seepage control is still an evolving and empirical engineering science. (U.S. Bureau of reclamation, 1991)

2.6.2.1 Rock Toe and Drains

The downstream toe of an embankment dam is the most critical region in respect of seepage instability. The entire seepage tends to concentrate around downstream toe, if internal drainage is not provided. The soil mass in this region is subjected to excessive seepage forces. This may cause heaving and sloughing of the toe if not properly protected. Rock toes, drainage blanket and filter drains are provided on the downstream of the dam to; (i) Provide a controlled outlet to seepage, (ii) to lower the seepage line and keep it within the downstream face and (iii) to prevent piping and heave at the downstream toe and thus improves the stability of the dam against seepage. For dams of low to moderate heights and where rock is available a rock toe of 1/4 to 1/3 the height of the dam can be provided. Since the quantity required in rock toe of a dam would be about 10% of the total quantity of fill required, the rock toe may be expensive in some situations. In order to check the migration of the particles from the shell and the foundation into the rock toe it has to be protected by the filters (Bharat Singh and Varshney, 1995, as cited in Fikadu, 2006

2.6.2.2 Horizontal Drainage

For dams of low to moderate height horizontal drainage blankets are used to drain the embankment as well as the downstream portion of foundation. The length of horizontal drainage depends on the flow- net; however, U.S.B.R recommended that the length of the blanket be equal to three times the height of the dam. Moreover, it must be sufficient cross-section to convey the maximum quantity of seepage estimated to come through the dam section since it is supplemented by the chimney drains on the upstream side and with toe drains on the downstream side. The suitable material for the construction of the horizontal drainage is the coarse material having high permeability. (Fikadu, 2006)

2.6.3.3 Chimney Drains

Chimney drains are the most important seepage controlling methods especially in the zoned embankment. Chimney drains intercepts all layers of the dam section in the seepage zone. Thus controls the seepage emerging in the downstream face of the dam. The chimney drains helps in reducing pore water pressure. Chimney drains may be vertical or inclined, upstream and downstream. An upstream inclined drain provided in an impervious fill makes the upstream portion behave more or less as a thin core. The water from a chimney drain has to be taken out through horizontal drains and a toe drain. The chimney drain has to be protected on all sides by filter layers. The advantage of providing chimney drains in the earth dam is to keep the downstream portion free from seepage when the reservoir is full. (Fikadu, 2006)

2.7 Stability Analysis Methods

2.7.1Fellenius' Method 1936

The Fellenius method, also known as the Swedish or Ordinary method of Slices is the first and most simple method of slices recorded in literature (Sivakugan & Das, 2009). The method assumes that the interstice forces are ignored and satisfies moment equilibrium only. Figure: 3 shows the forces on an individual slice with the forces shown in red assumed negligible.

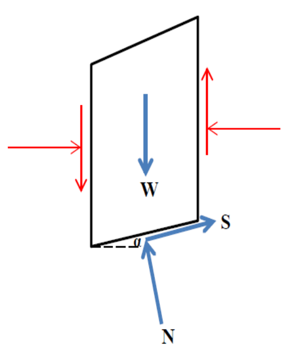


Figure2. 2: Free body diagram of ith slice- Fellenius method

Factor of safety in terms of total stress:

$$F = \frac{\Sigma\,(cL + W\cos\alpha\tan\phi)}{\Sigma\,W\sin\alpha} \qquad \text{2.1}$$

Factor of safety in terms of effective stress:

$$F = \frac{\Sigma\,(c'L + (W\cos\alpha - u\,L)\tan\phi')}{\Sigma\,W\sin\alpha} \qquad \text{2.2}$$

The factor of safety can be hand calculated due to its simplicity. The GEO-SLOPE Stability Modeling (2008) guide recommends that the Fellenius method "should not be used in practice, due to potential unrealistic factors of safety" this is because the method underestimates factors of safety as it ignores the effects of any interslice forces. The Fellenius method has been used in this thesis for comparative purposes only and the baseline case analyses demonstrate the differences (often significant) between Fellenius' method and other methods that do take into account interslice forces.

2.7.2Bishop's simplified method 1955

Bishop's simplified method assumes that there are no interslice shear forces, only interslice normal forces acting horizontally on the slice (Bishop, 1955).

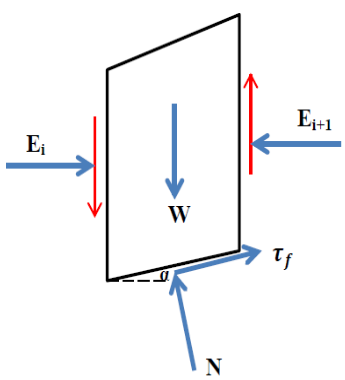


Figure2. 3: Free body diagram of ith slice- Bishop's simplified method

The derivation for factor of safety using Bishop's simplified method is shown below (Sivakugan and Das (2009); Duncan and Wright (2005)). Equilibrium of forces in the vertical direction (positive is upwards) and rearranging for *N*:

$$N = \frac{\mathrm{W} - \tau \mathrm{fsin}\alpha}{\cos\alpha} \quad \text{.......... 2.3}$$

$$F = \frac{c\prime \mathrm{L} + (\mathrm{N} - \mathrm{u\ L})\tan\phi\prime}{\mathrm{F}} \quad \text{.......... 2.4}$$

2.7.3Morgenstern and Price 1965

The Morgenstern and Price (1965) method considers limit equilibrium of both force and moment for each slice in circular and non-circular slip surfaces. The method assumes a relationship between the interslice forces (X and E) with a function ((x)) that varies continuously across the failure surface and an unknown scaling factor (λ):

$X=\lambda \times (x) \times E$ ……………………………………………………………......……2.5

The force function can be constant (same as Spencer's method), half-sine, trapezoidal or data-point specified (GEO-SLOPE International Ltd, 2008).

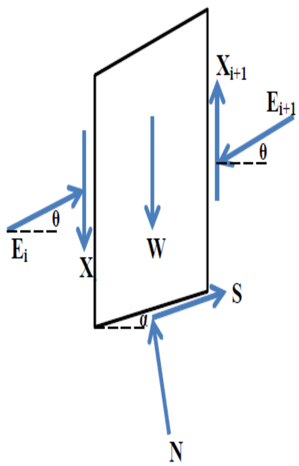


Figure2. 4: Free body diagram of ith slice- Morgenstern and Price method

This equation has too many unknowns so alternatively two factors of safety are calculated, one from moment equilibrium, and the other from force equilibrium.

$$F = \frac{\Sigma(c'L+(N-u\,L)\tan\phi')}{\Sigma(W-(Xi+1-Xi))\tan\alpha+\Sigma(Ei+1-Ei)} \text{...} 2.6$$

The software analyses large number of slip circles based on the methodology developed by Fellenius, Bishop, Morgenstern - Price and etc. gives the minimum factor of safety for the critical failure surface. The method developed by Morgenstern - Price considers both for the horizontal force equilibrium (Ff) and moment equilibrium (Fm) for giving the minimum factory of safety. His method is considered preferable as the moment equilibrium on individual slice is used to calculate interslice shear force.

2.8 Design Consideration of Stability Analysis

The various design conditions of analysis for upstream and downstream slope along with the minimum values of factors of safety to be aimed at and use of type of shear strength for each condition of analysis has been provided. IS code No 7894 -1975 which is reproduced in the following table:

Table2. 2: Minimum Desired Values of Factors of Safety and Type of Shear Strength for Various Loading Conditions

Case No	Loading Condition of Dam	Slope Most Likely to be Critical	Minimum Desired Factor of Safety
I	Construction condition with or without partial pool	Upstream and downstream Upstream	1.0
II	Reservoir partial pool	Upstream	1.3
III	Sudden draw down Maximum head water to minimum with tail water at maximum	Upstream Downstream	1.3 1.3
IV	Steady seepage with reservoir full	Downstream	1.5
V	Steady seepage with sustained rainfall	Downstream	1.3

Source :(OWWDSE report, 2007)

3 Research Methodology

3.1 Study Area

Arjo-Dhedhessa Irrigation Project is proposed on the Dhdhessa River, which is the largest tributary of Abbay (Blue Nile) river. The Abbay basin is by several criteria the most important river basin of Ethiopia. It accounts for almost 20 percent of Ethiopia’s land area; 50 percent of its total average rainfall; 25 percent of its population; 39 percent of national cattle herd; and over 40 percent of cultivated land and crop production. The Abbay River itself has an average annual run-off of about 49 BCM. The rivers of Abbay basin contribute about 62 percent of Nile total at Aswan.

The basin is a key surplus food producing area of Ethiopia. It is, therefore, critically important in terms of national agricultural economy and for national food security. Arjo-Dhedhessa Irrigation Project assumes importance in this background.

3.1.1 Location

The project area is located in East Wollega, Illubabor and Jima Administrative Zones of Oromiya National Regional State. It falls between latitude 8^0-30' 00 "and 8^0-40'-00" N and longitude 36^0.22'-00" and 36^0.43' 00"E. The project area is about 480 km from Addis Ababa through Jima and Bedele.

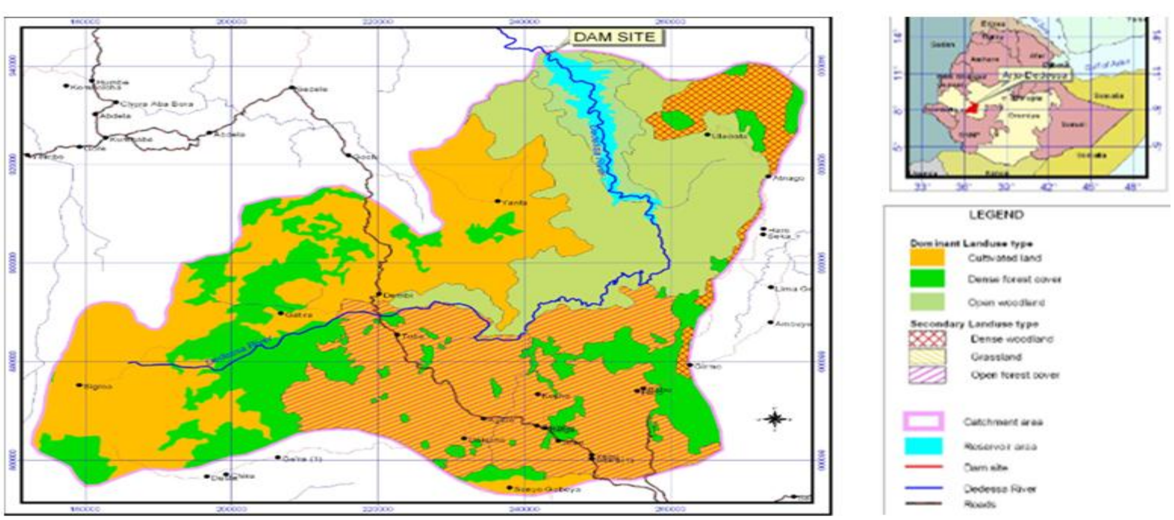


Figure3. 1: Location of the study area
Source: Arjo Dhidhessa Design Report, 2007

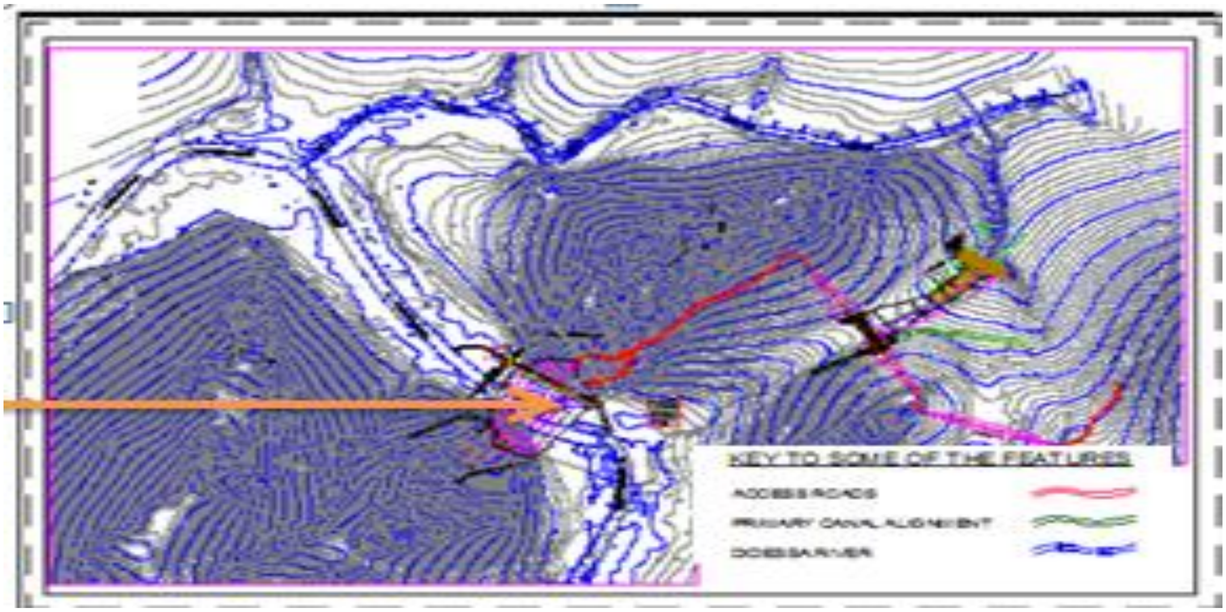


Figure3. 2: General Layout of the Dam
Source: Arjo Dhidhessa Design Report, 2007

3.1.2 Project Description

Considering the topography and the foundation rock formation, an earth fill dam section with an impervious clay core has been provided. Construction materials for an earth fill dam are available near the dam site area. A clay blanket has been provided in the river bed and at abutment on the upstream. The dam has been provided with the positive cut off trench up to rock foundation with side slopes cut off trench of 1H:1V and grout curtain with grout cup. Upstream surface slopes of the main dam and saddle dam have been considered to have a slope of 3.5 H:1V with berm width of 6.0 m at 15.0 m height interval up to ground level and likewise downstream slope with berms of 6.0 m width at 15.0 m height interval up to ground level. The downstream face slope of 2.5 H:1V has been consider. (OWWDSE, 2009 Arjo Dhidhessa dam report)

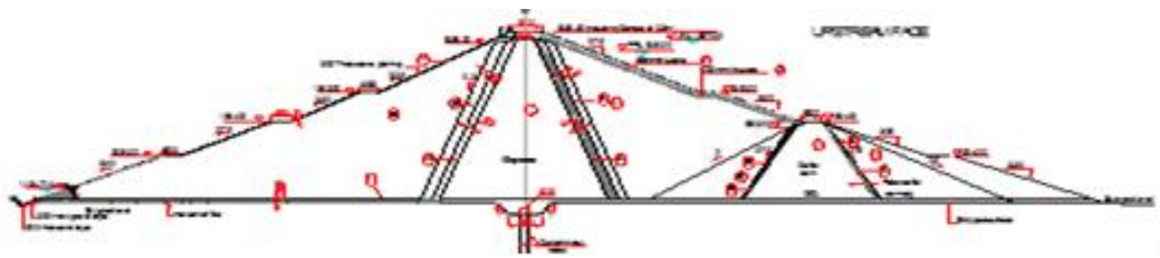


Figure3. 3: Arjo Dhidhessa main dam cross-sectional
Source: Arjo Dhidhessa Design Report, 2007

3.1.3Arjo Dhidhessa Dam site Geology

The dam site is characterized by two relatively steep hills called as abutments and a relatively flat river channel in between these abutments. Alkaline vertical dikes cover the slope and top of abutments, which gave steep land form. The volcanic rocks are exposed in creeks, abutments, river bed and hills of the projects area. Further downstream of the dam axis metamorphic rocks are out cropping. The left abutment for investigating the ground conditions, which indicate alkaline volcanic rocks, composed of alkaline basalt, trachyte basalt with vesicular nature and aphanatic basalt covering abutment. The alkaline volcanic rocks exposed at dam axis occur as dyke and align in NE direction. The alkaline volcanic rocks on exposure become highly weathered. No fresh rock had been encountered even by drilling up to 40.0 m depth.

The valley floor is covered with alluvial deposit of 3 to 5 meter depth. Basaltic trachyte occurs under the alluvial deposit and the river bed. The formation is fractured up to 20 meter depth and is slightly to moderately weathered and strong. Two strike slip faults / lineaments are observed at the foot of both abutments. The core recovery from the bore holes drilled on the valley floor are sheared and filled with white precipates. The rock formation below the basaltic trachyte is solid, moderately strong and slightly weathered with radial porphyritic texture. The right abutment indicates the formation to be uniform of alkali volcanic rock. The rocks are strong moderately weathered and fractured.

Figure3. 4: Rock Outcrop at River bed Upstream of Dam site

3.1.4 River Diversion

The river diversion arrangement during construction have been considered by construction of a coffer dam on the upstream side of 20.0 m height which would ultimately form an integral part of the main earth and rock fill dam and a D/s coffer dam 50 m away from the D/s of the dam and about 6.0m high to provide working area free from water. RCC twin duct of size 3.5 m wide x 3.5 m high would be constructed on the right abutment. The river bed level is at EL 1312.0 m. Therefore in order to prevent silt entering the duct the invert level of the duct has been provided at EL, 1314.0m, i.e. 2.0 m above the river bed.

Figure3. 5: Diversion Conduit

3.1.5construction Program

The Project Office was opened during December 2005 with the Project Manager. The Office furniture and equipments were procured, requisite staff members were engaged and office started functioning during December 2005. The Team Leader along with the Dam Engineer (Foreign) arrived from India on the 3rd February 2006. Immediately after their arrival they along with local consultants for Dam Design Group undertook a visit to the Project sites during February 7-14, 2006. Other foreign and local consultants were mobilized subsequently.

During the lean period of 1st year when the discharge in the river bed is very low, the excavation of the cut off trench up to design level in the left abutment reach for a length of 125.0 m (211.0 m to 336.0 m) and right abutment for a length of 100.0 m (From 620.0 m to 720.0 m) involving total length of 225.0 m is proposed to be done. Consolidation grouting, providing concrete cap on the grout hole tops and back filling of trench is to be taken up simultaneously with this excavation. Simultaneously with the construction of the RCC twin barrel duct, its construction and completion is to be done. The site clearance for the cut off trench, shell zone and construction of the rock fill dam portion up to EL 1340.6 is to be achieved. Site clearance on the upstream of the dam axis up to toe of the dam, raising the main cofferdam which is ultimately to form the integral part of the main dam up to EL 1340.6 along with upstream protection works is to be completed. In the 2nd year work on the downstream for site clearance, core filters, horizontal filter, rock toe and downstream shell may also be taken up after construction of the above, the routed flood discharge of 100 year return period which equal 215.0 m^3/sec shall be diverted through the completed twin barrel box duct. In the 3rd year of working season period remaining works, relating to consolidation grouting, curtain grouting, caps, filter and shell reach are to be taken up and the normal raising of the earth and rock fill dam, impervious core, filter up to maximum possible elevation feasible should be done. During 4th year the balance works of curtain grouting and raising up of the dam up to design height, providing rock rip rap on the upstream slope, construction of wave wall, D/S drainage and protection of slope are to be completed. This actually is to performed in the last three month of the working reason after the dam construction is completed up to crest level. Spillway structure and facilities in saddle ADS-1, Irrigation outlet and intake structure in saddle ASD-1 for Right Bank canal and the Saddle dam are all independent and the construction of these does not interfere with other components of the work. These as such shall be taken up simultaneously from the very start of the construction works of main dam. The construction work for the irrigation out let and its intake structure for the left bank primary canal likewise is to be taken up independently along with other works from the start of the project. (OWWDSE main report, May 2007)

3.1.6Executed Works

The contractor (OWWCE) has completed the following dam part activities:

- 2013 foundation excavation
- 2014 diversion conduit
- 2015 cofferdam construction up to 1334 elevation and grouting

3.2 Methods

3.2.1 Data Collection

These were data used in this thesis:

- permeability
- reservoir water level for both cofferdam and main dam
- dam height, width (for bottom and top)
- properties of selected material
- reservoir capacity
- peak rate of bottom outlet out flow
- bottom outlet design
- MDDL of the reservoir

The above data were collected from Oromiya Water Works Design and Supervision Enterprise (OWWDSE) and Water Works Design and Supervision Enterprise (WWDSE).

3.2.1.1 Primary Data

Primary data were collected from Arjo Dhidhessa Embankment dam site. During site visits cross checking of secondary data that already have been collected from Oromiya Water Works Design and Supervision Enterprise and Water Works Design and Supervision Enterprise:

- Interview of the resident engineers, site engineers and surveyors
- Collection of photos that show cofferdam, reservoir level, main dam trench excavation, grouting area.

3.2.1.2 Secondary Data

Arjo Dhidhessa Embankment dam Design document review has been taken from WWDSE and OWWDSE.

3.2.2 Model Approach

The flow of water through soil is one of the fundamental processes in geotechnical and geo-environmental engineering. In fact, there would little need for geotechnical engineering if water were not present in the soil. Seepage is used to describe all movement of water through soil regardless of the creation or source of the driving energy or whether the flow is through saturated or unsaturated soils. Simulating the flow of water through soil with a numerical model can be very complex. Natural soil deposits are generally highly heterogeneous and non-isotropic. In addition, boundary conditions often change with time and cannot always be defined with certainty at the beginning of an analysis.

While, SEEP/W to do seepage analyses, it is also about general numerical modeling techniques. Numerical modeling, like most things in life, is a skill that needs to be acquired.

3.2.2.1 Model Selection

The unprecedented computing power now available has resulted in advanced software products for engineering and scientific analysis. The ready availability and ease-of-use of these products makes it possible to use powerful techniques such as a finite element analysis in engineering practice. These analytical methods have now moved from being research tools to application tools. This has opened a whole new world of numerical modeling.

Software tools are an extremely powerful calculator, obtaining useful and meaningful results from this useful tool depends on the guidance provided by the user. It is the users' understanding of the input and their ability to interpret the results that make it such a powerful tool.

GeoStudio2007 is finite element method which is now commonly adopted for modeling civil engineering structures mainly:

- Embankment Dams
- Reinforced walls and slopes
- Excavation and open pit mines
- Roads and bridges
- Earthquake deformation

It is a powerful tool for the analysis of many geotechnical structures. The sub program of GeoStudio2007 software used in seepage modeling in embankment dams, ground water seepage analysis and for excess pore water pressure dissipation problems, slope modeling in embankment dam.

3.2.2.2 Model Application Approach

3.3.2.2.1 Seepage Modeling In SEEP/W Computer Program

SEEP/W is numerical modeling software which used to solve the practical seepage problems. This is a part of the most popular geotechnical software called Geo-studio. The SEEP/W program is created with the combination of seepage theory and finite element method and working on saturated/unsaturated soil region.

The practical seepage problems are never easy to convert into a numerical modeling because of the heterogeneity of the natural soils and the varying boundary condition. Generally the boundary conditions for a seepage problem never being as same as found in the initial stage. Therefore the seepage analysis in SEEP/W program is divided in to two categories.

1. Steady-state analysis

In the steady state the fundamental water flow properties such as water pressure and water flow rates never going to be changed. Practically achieving steady state is impossible. The purpose of the steady-state analysis is only to know how the initial input parameters respond to a given boundary condition.

This analysis never state that how long it takes to reach a steady state. It returns as set of solved values for water pressure and water flow parameters for particular boundary conditions. A constant pressure (H) and a constant flux rate are the important boundary condition used for a steady-state analysis.

2. Transient analysis

   Transient analysis is used to know how long the embankment takes to responds for a given boundary condition. Therefore the fundamental flow properties (pressure and water flow rate) will vary with time. The analysis required an initial boundary condition as well as a destination boundary condition.

The SEEP/W program has ability to read the initial condition from another analysis (may be SEEP/W) and generally obtained from a steady-state analysis (John 2010).

3.2.2.2.2 SLOPE/W (Geo Studio)

SLOPE/W is the most common and popular software application which used for the stability analysis of a slope. This is a part of Geo studio software application. This application is created based on limit equilibrium method and included several types of methods like Fellenius, Bishop and Morgenstern-price method (sivakugan and Das 2009). The stability analysis using SLOPE/W included following components (krahn 2004).

1. Drawing geometry.
2. Defining soil properties and assigning for the corresponding soil layer.
3. Defining the water table.
4. Selection of analysis method.
5. Problem solving and display the results.

The result of stability analysis from the SLOPE/W can be obtained as obtained as both visuals numbers. The visually interpreted results make it possible to easy understand of the results in numbers. The very important advantage of the SLOPE/W analysis is it allows handling all possible slides in a same model with the corresponding factor of safety.

4 Result and Discussion

4.1 Seepage and Stability Analysis

4.1.1 Designed Cofferdam Seepage Steady-State Analysis

First, normal condition of coffer dam analyzed with steady-state seepage analysis and the initial condition for transient analysis was obtained. Boundary conditions are defined by the total head along the upstream slope, zero pressure at the toe of the downstream slope and the potential seepage.

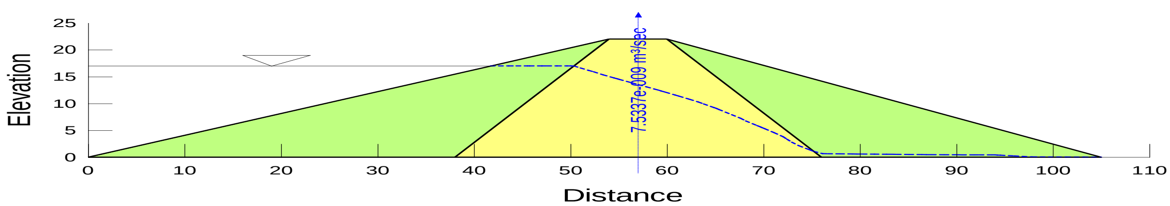
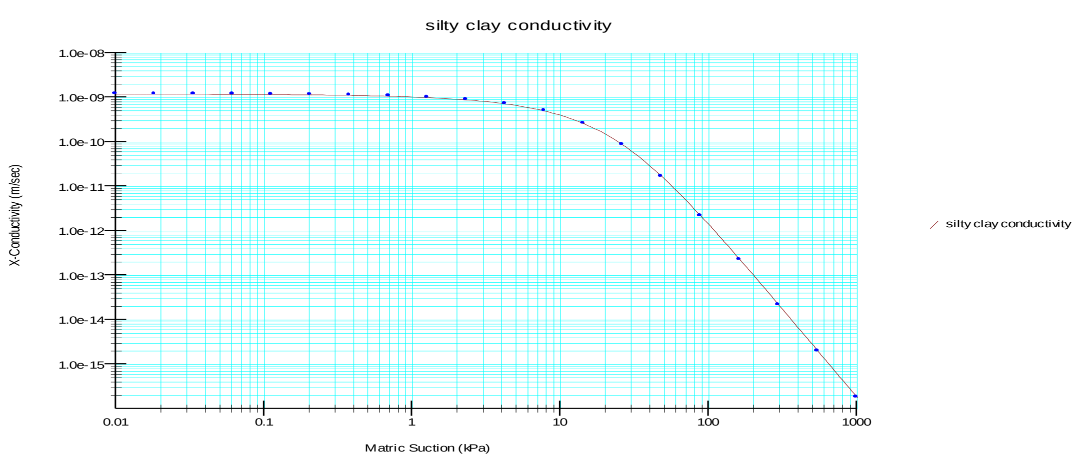
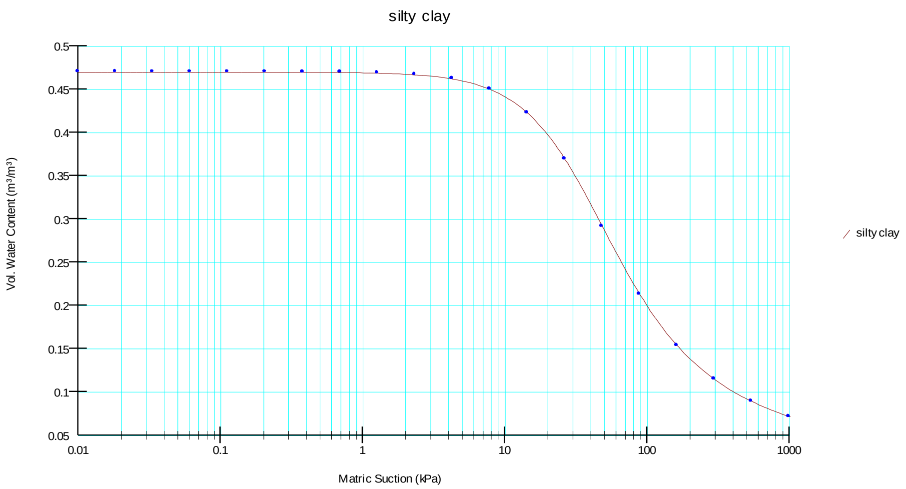


Figure4. 1: Designed Cofferdam steady-state seepage analyses

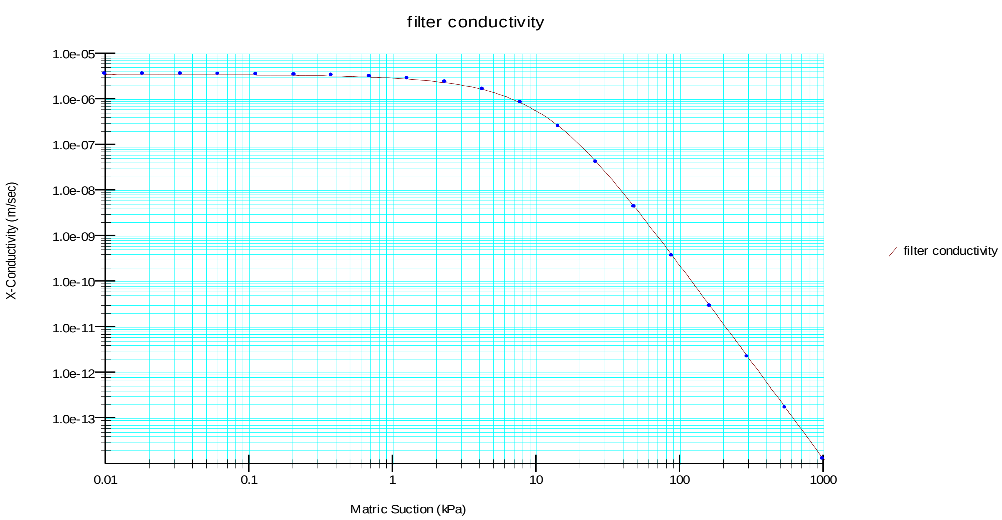
In steady-state analysis, the total flux through the cross section is **7.5575*10^{-9} m^{3}/s.**



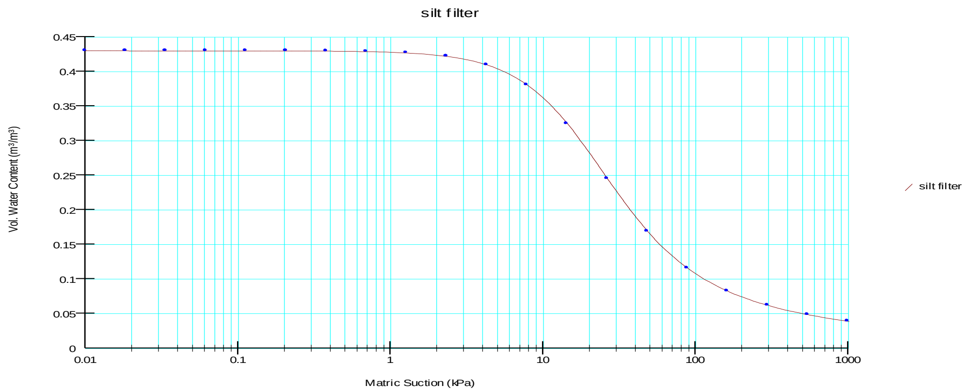
a) Hydraulic conductivity Function of silty clay



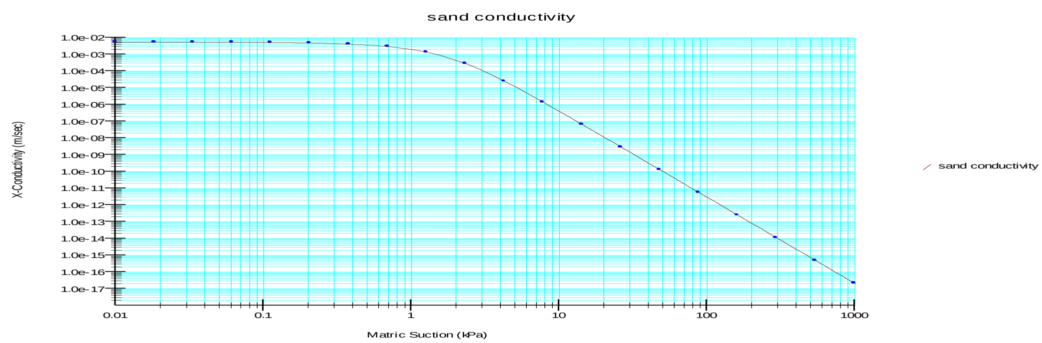
b) Volumetric water content of silty clay



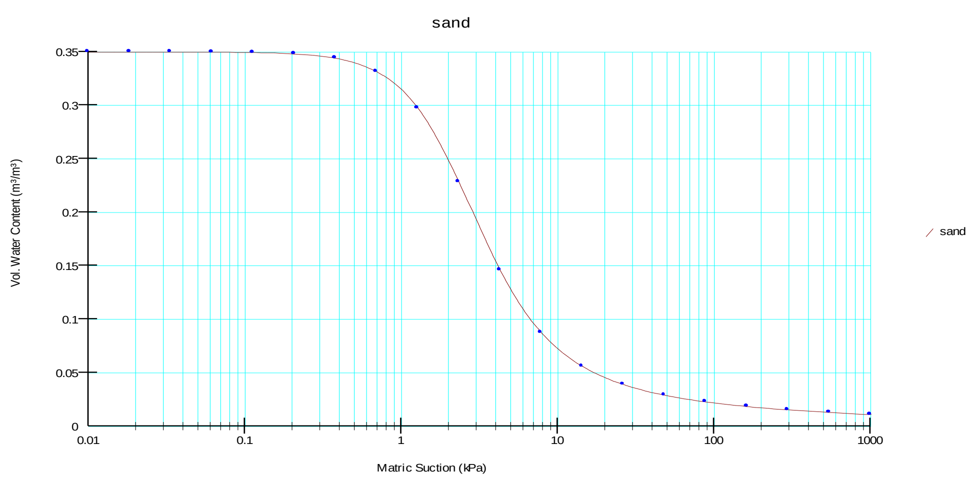
c) Hydraulic conductivity of filter



d) Volumetric water content of silt filter



e) Hydraulic conductivity of sand



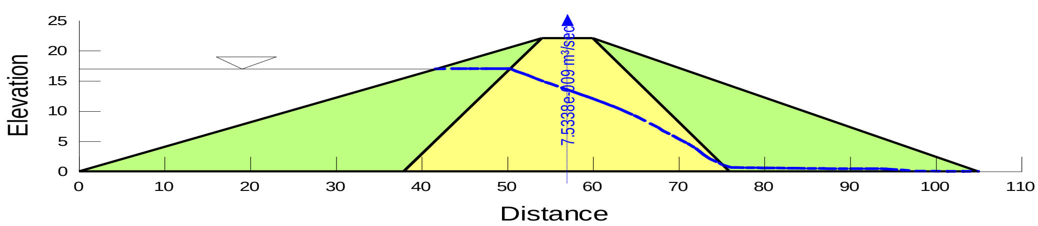
f) Volumetric water content of sand

Figure4. 2: dam material function

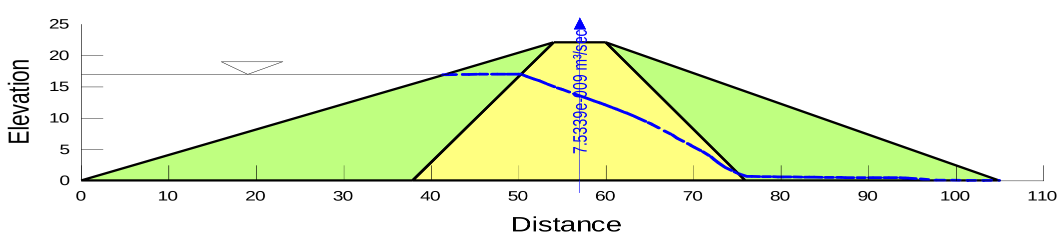
4.1.2 Designed Cofferdam Transient Seepage Analysis

During the drawdown the water level of the coffer dam reduced from 17m to 0m. But, the figure below shows only the full supply level. The transient analysis could be done based on the steady-state analysis as parental analysis. Therefore the pressure head and the pore water pressure which obtained from the steady-state analysis are transferred to the transient analysis as the boundary condition. The properties of soil such as permeability and the volumetric water content which defined in steady-state analysis also imported to the transient analysis.

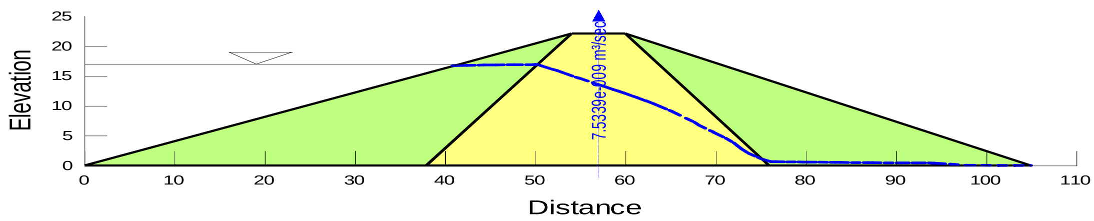
Initially the time duration for analysis was defined as 30 days with 10 time steps and the time increment was exponential manner. Every time steps in the model was saved and taken as the results.



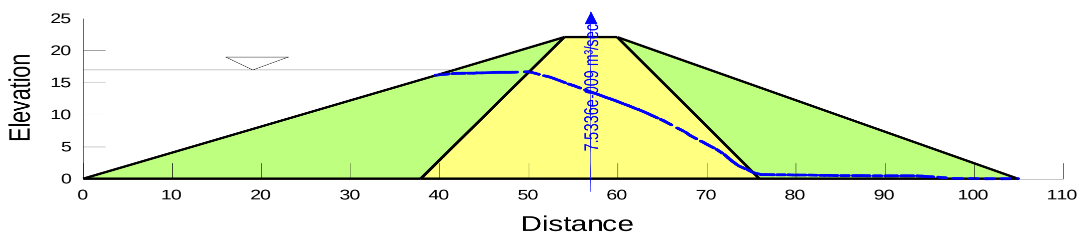
a) Designed Cofferdam seepage conditions at 4.8 hr of drawdown.



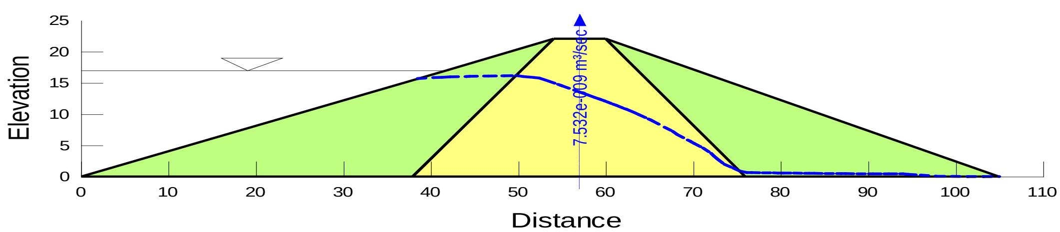
b) Designed Cofferdam seepage conditions at 12.3 hr of drawdown.



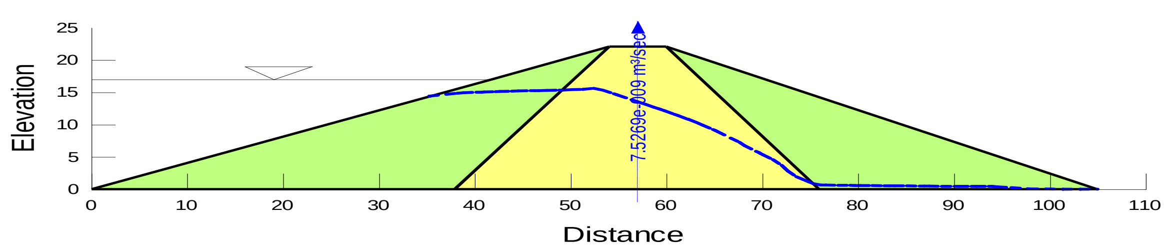
c) Designed Cofferdam seepage conditions at 24 hr of drawdown.



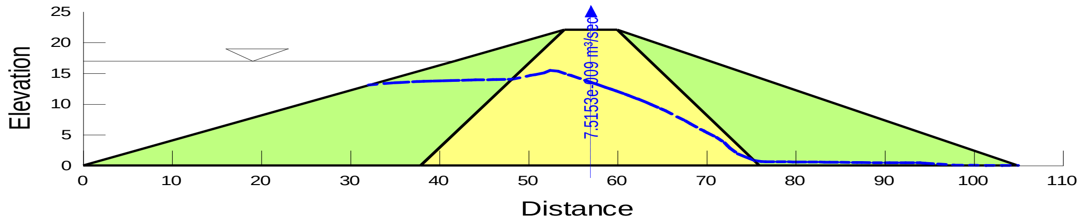
d) Designed Cofferdam seepage conditions at 1.76 days of drawdown.



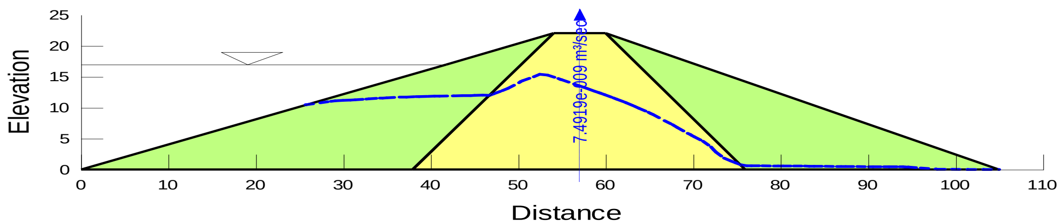
e) Designed Cofferdam seepage conditions at 2.94 days of drawdown.



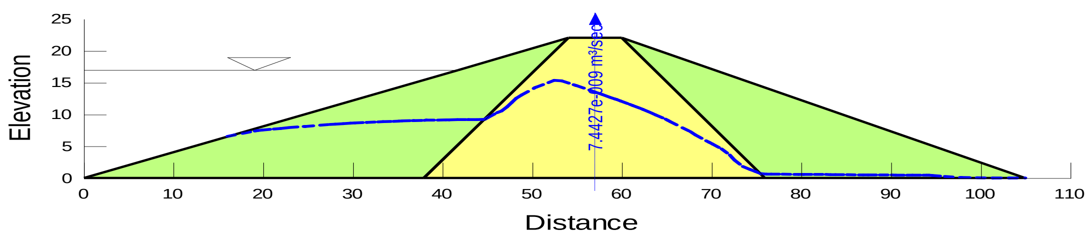
f) Designed Cofferdam seepage conditions at 4.78 days of drawdown.



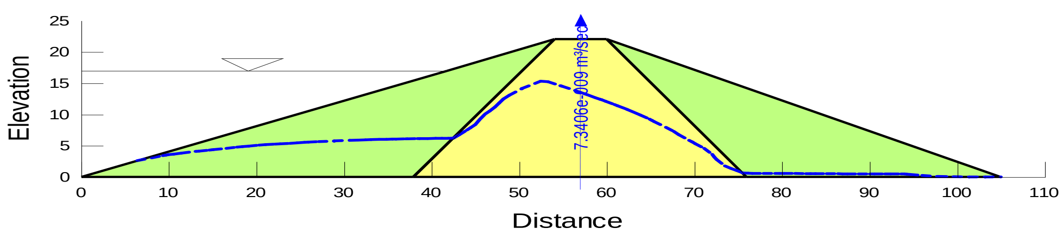
g) Designed Cofferdam seepage conditions at 7.65 days of drawdown.



i) Designed Cofferdam seepage conditions at 12.1days of drawdown.



j) Designed Cofferdam seepage conditions at 19.1 days of drawdown.



k) Designed Cofferdam seepage conditions at 30 days of drawdown

Figure4. 3: Designed Cofferdam Transient Seepage Condition

Figure 4.2 (a-k) shows the seepage conditions for various time periods during drawdown. The water table becomes decreased from 17m to 0m.The cofferdam total fluxes changes summarized in the following table 4.1.

Table4. 1: Designed cofferdam total flux changes with increase time

Time	Seepage through the dam($10^{-9}m^3/s$)
4.8 hr	7.5338
12.3 hr	7.5339
24 hr	7.5339
1.76 day	7.5336
2.94 days	7.5320
4.78 days	7.5269
7.65 days	7.5153
12.1 days	7.4919
19.1 days	7.4427
30 days	7.3406

The figure 4.2(a-k) shows varies stages of transient seepage analysis. From the initial stage, the total flux through the coffer dam is decreasing with increasing time. This shows the coffer dam has no seepage problem.

4.2 Designed Cofferdam Stability Analysis

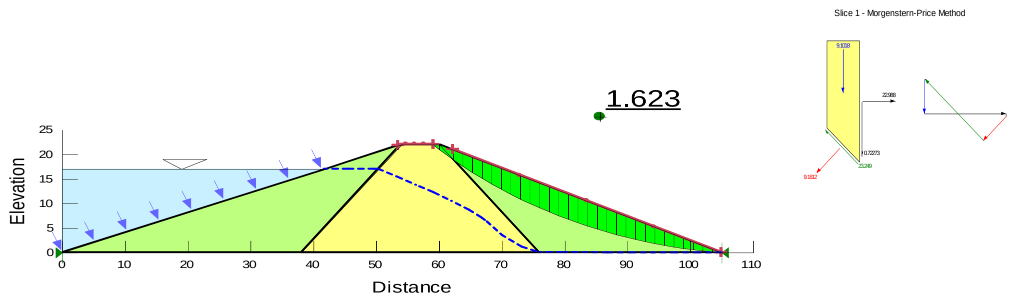
4.2.1Downstream Designed Cofferdam Stability Analysis

Stability analysis has been done with Mohr-coulomb method and the strength parameters defined as follows:-

For core material (silty clay): unit weight=18.4KN/m^3, cohesion=30kpa, phi=26^0

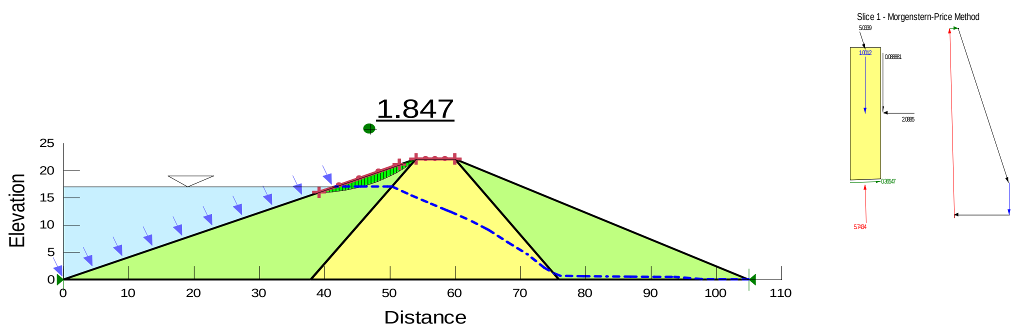
For rock fill: unit weight=20.5KN/m^3, cohesion=0.5kpa, phi=35^0

Factor of safety is calculated using Morgenstern-price method.



a) steady-state d/s side free body diagram and force polygon Morgenstern-price method

4.2.2 Upstream Designed Cofferdam Stability Analysis



b) steady-state u/s side free body diagram and force polygon Morgenstern-price method

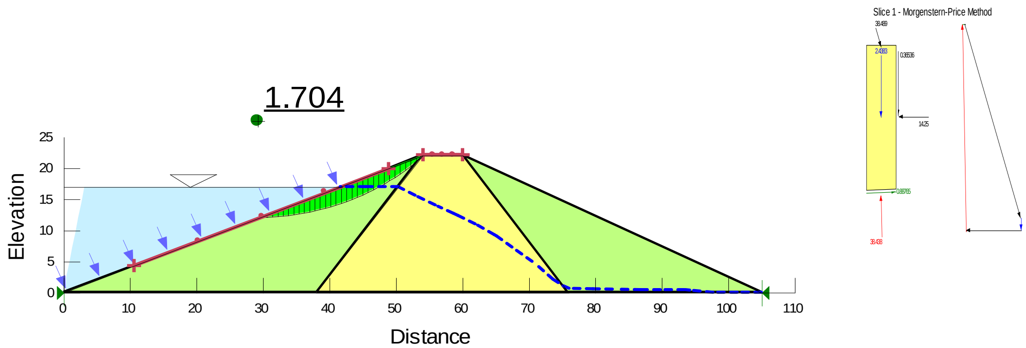
Figure4. 4: Designed Cofferdam Slope Stability during Steady- State

Table4. 2: U/S and D/S cofferdam factor of safety

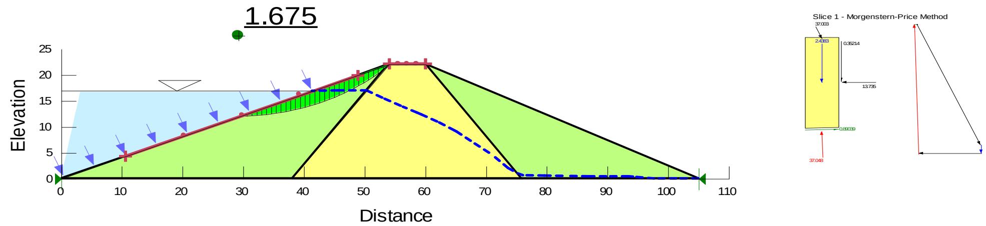
Condition	U/S	D/S	Method
Steady-state	Factor of safety obtained		
	1.847	1.623	Morgenstern-price method

The stability analysis of both U/S and D/S slopes of the cofferdam have been done with the properties of shell material and clay material. The factors of safety for upstream 1.758 and factor of safety for downstream slope is (1.623) so, the slope has enough stability during steady-state. (FOS ≥1.3)

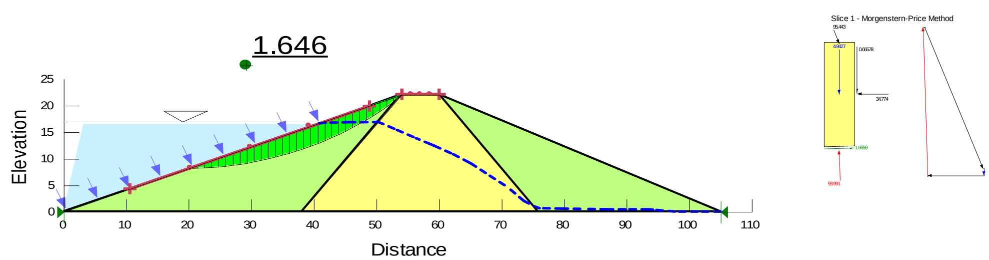
4.2.3 Designed Cofferdam Drawdown Slope Stability Analysis



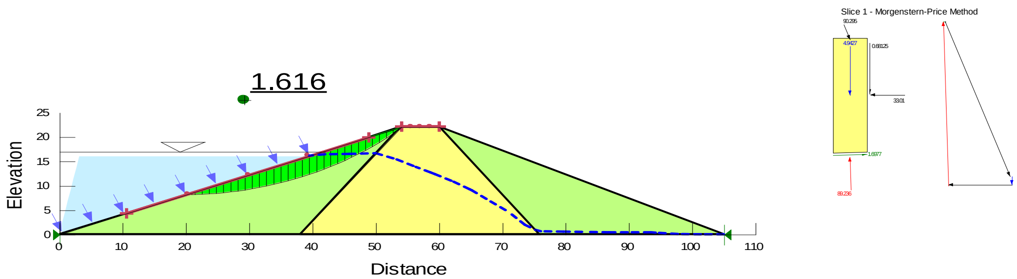
a) Designed cofferdam stability analysis after 4.8 hr with free body diagram and force polygon



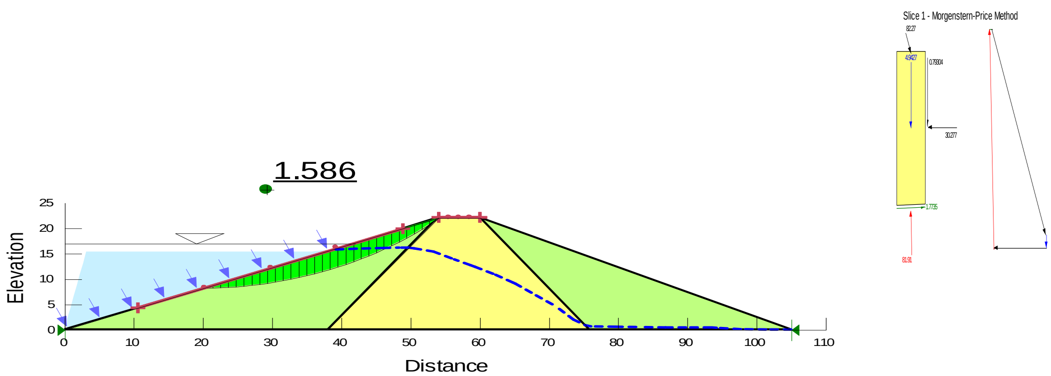
b) Designed cofferdam stability analysis after 12.3 hrs with free body diagram and force polygon



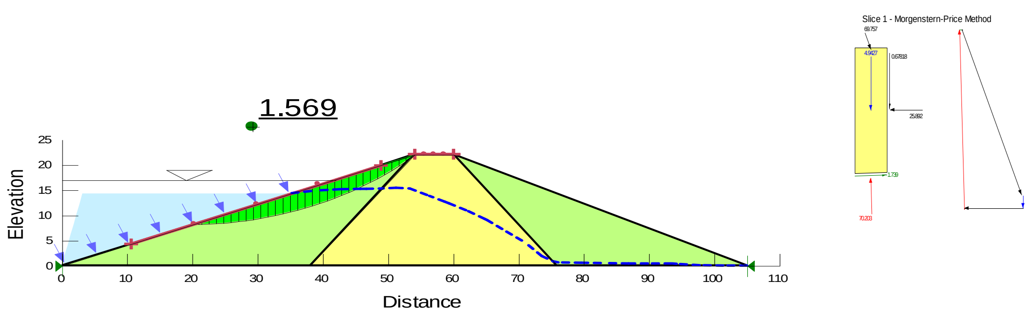
c) Designed cofferdam stability analysis after 24 hrs with free body diagram and force polygon



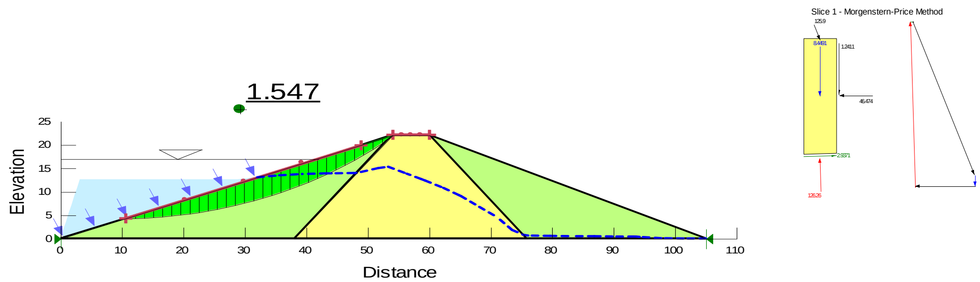
d) Designed cofferdam stability analysis after 1.76 days with free body diagram and force polygon



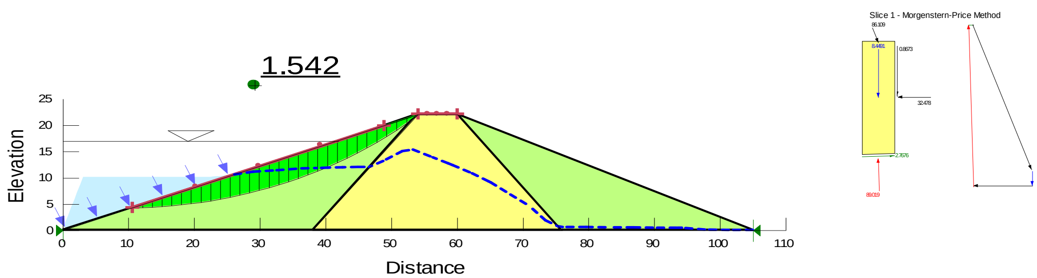
e) Designed cofferdam stability analysis after 2.94 days with free body diagram and force polygon



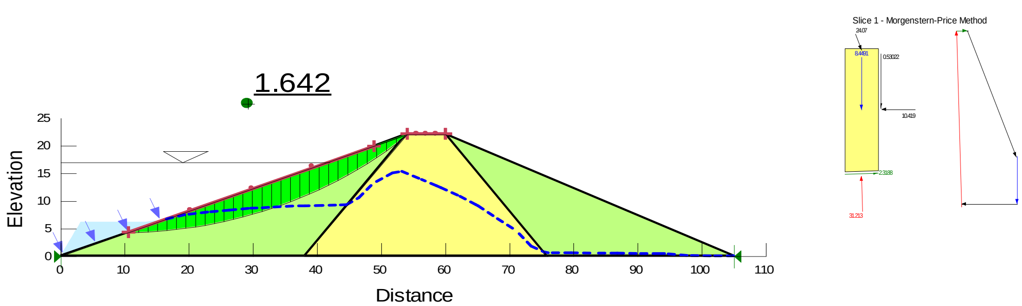
f) Designed cofferdam stability analysis after 4.78 days with free body diagram and force polygon



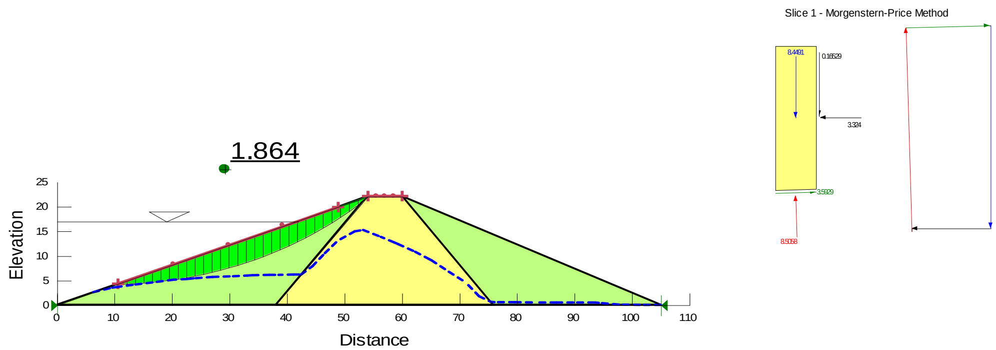
g) Designed cofferdam stability analyses after 7.65 days with free body diagram and force polygon



h) Designed cofferdam stability analysis after 12.1 days with free body diagram and force polygon



i) Designed cofferdam stability analysis after 19.1 days with free body diagram and force polygon



j) Designed cofferdam stability analysis after 30 days with free body diagram and force polygon

Figure4. 5: transient analysis of u/s side free body diagram and force polygon Morgenstern-price method

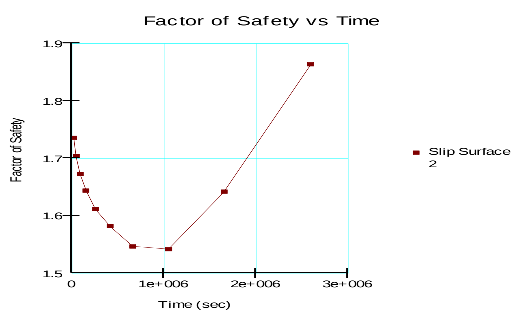


Figure4. 6: Factor of Safety Vs Time of Designed Cofferdam

Table4. 3: Designed cofferdam drawdown stability analysis

Time	Safety factor	Method
4.8 hr	1.704	Morgenstern-price method
12.3 hr	1.675	Morgenstern-price method
24 hr	1.646	Morgenstern-price method
1.76 day	1.616	Morgenstern-price method
2.94 days	1.586	Morgenstern-price method

4.78 days	1.569	Morgenstern-price method
7.65 days	1.547	Morgenstern-price method
12.1 days	1.542	Morgenstern-price method
19.1 days	1.642	Morgenstern-price method
30 days	1.864	Morgenstern-price method

The factor of safety is decreases and then increasing until the end of the analysis. The results show that the slope is potentially stable throughout the drawdown. It is because of the obtained value meet the minimum required. (FOS ≥1.3)

4.3Constructed Cofferdam Seepage Steady-State Analysis

First, normal condition of coffer dam analyzed with steady-state seepage analysis and the initial condition for transient analysis was obtained. Boundary conditions are defined by the total head along the upstream slope, zero pressure at the toe of the downstream slope and the potential seepage.

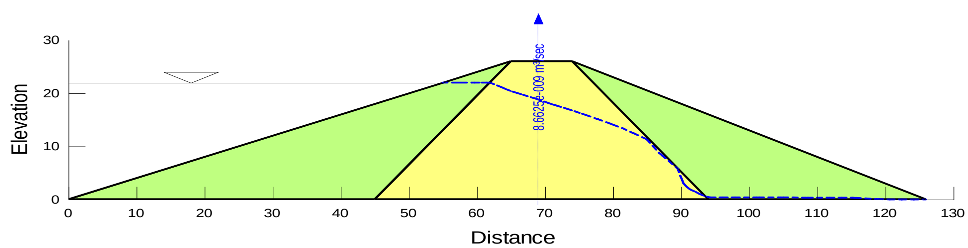


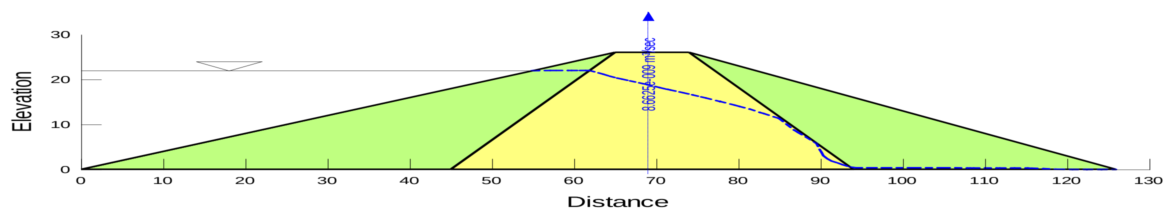
Figure4. 7: Constructed Cofferdam steady-state seepage analyses

In steady-state analysis, the total flux through the cross section is **8.6625*10^{-9} m^{3}/s.**

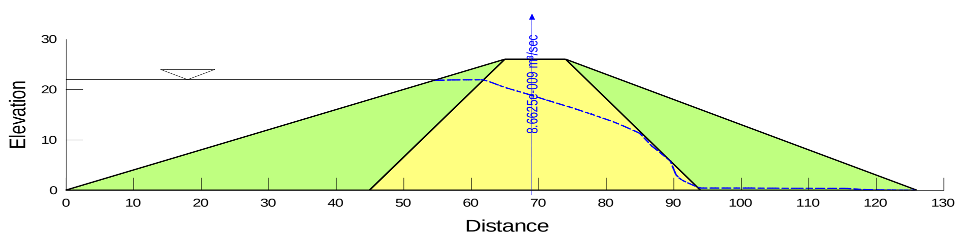
4.3.1 Constructed Cofferdam Transient Seepage Analysis

During the drawdown the water level of the coffer dam reduced from 22m to 0m. But, the figure below shows only the full supply level. The transient analysis could be done based on the steady-state analysis as parental analysis. Therefore the pressure head and the pore water pressure which obtained from the steady-state analysis are transferred to the transient analysis as the boundary condition. The properties of soil such as permeability and the volumetric water content which defined in steady-state analysis also imported to the transient analysis.

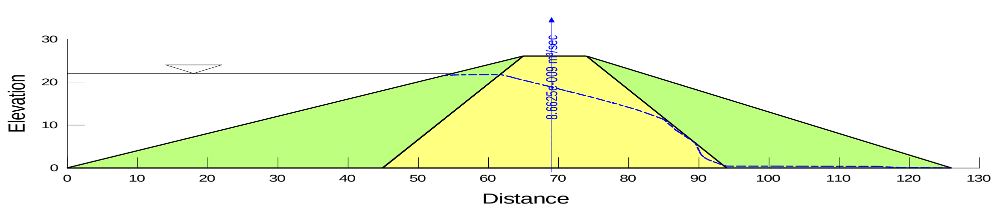
Initially the time duration for analysis was defined as 30 days with 10 time steps and the time increment was exponential manner. Every time steps in the model was saved and taken as the results.



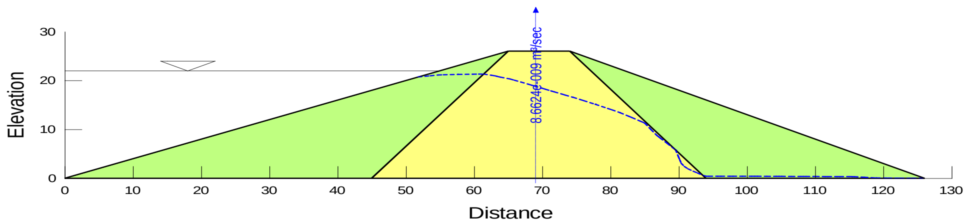
a) Constructed Cofferdam seepage conditions at 3.11 hr of drawdown.



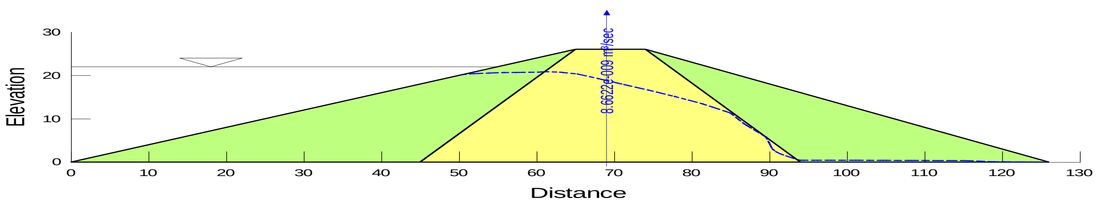
b) Constructed Cofferdam seepage conditions at 7.69 hr of drawdown.



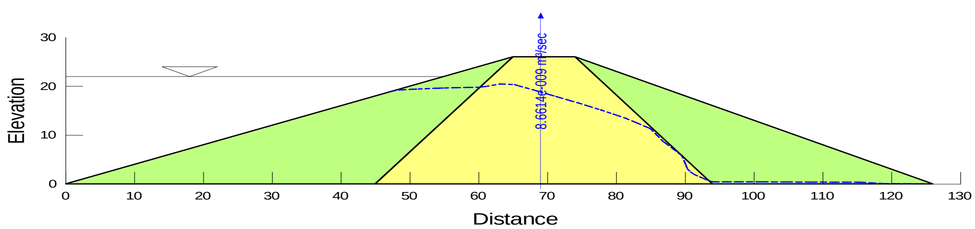
c) Constructed Cofferdam seepage conditions at 14.4 hr of drawdown.



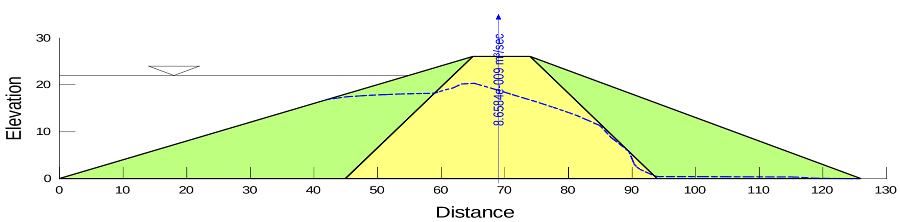
d) Constructed Cofferdam seepage conditions at 1.02 days of drawdown.



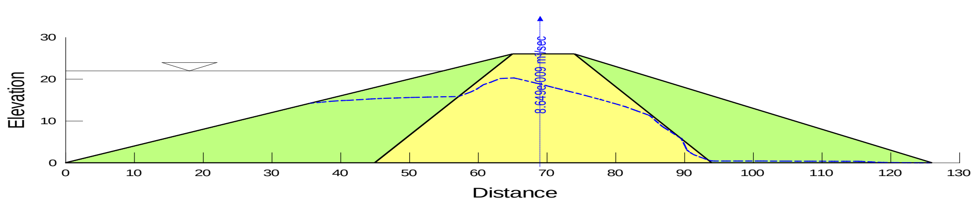
e) Constructed Cofferdam seepage conditions at 1.63 day of drawdown.



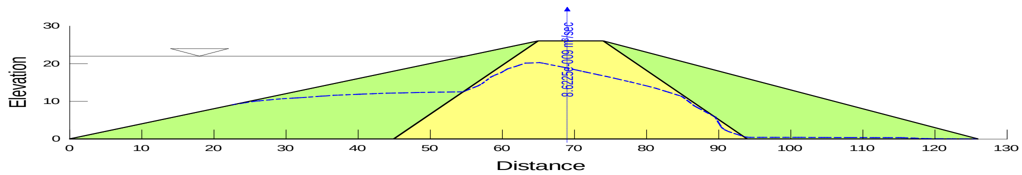
f) Constructed Cofferdam seepage conditions at 2.53 days of drawdown.



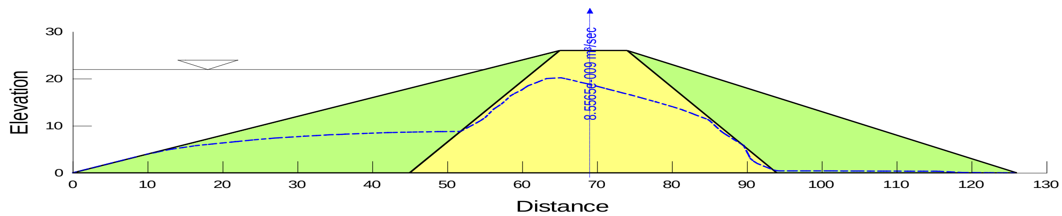
g) Constructed Cofferdam seepage conditions at 3.53 days of drawdown.



h) Constructed Cofferdam seepage conditions at 5.81 of drawdown.



i) Constructed Cofferdam seepage conditions at 8.7 days of drawdown.



j) Constructed Cofferdam seepage conditions at 30 days of drawdown

Figure4. 8: constructed cofferdam transient seepage analysis

Figure 4.2 (a-k) shows the seepage conditions for various time periods during drawdown. The water table becomes decreased from 22m to 0m.The cofferdam total fluxes changes summarized in the following table 4.4.

Table4. 4: Constructed cofferdam total flux changes with increase time

Time	Total seepage through the dam($10^{-9}m^3/s$)
4.8 hr	8.6625
12.3 hr	8.6625
24 hr	8.6625
1.76 day	8.6624
2.94 days	8.6622
4.78 days	8.6614

7.65 days	8.6584
12.1 days	8.649
19.1 days	8.6225
30 days	8.5565

The figure 4.2(a-k) shows varies stages of transient seepage analysis. From the initial stage, the seepage (total flux) through the coffer dam is decreasing with increasing time. This shows the coffer dam has no seepage problem. But excessive seepage occurred currently the coffer dam of the Arjo Dhidhessa. Excessive seepage emerged from foundation, abutments and near the conduit. During the overtopping occurred, water has stored in the abutments. When the foundation core excavated, water come to the core foundation by gravity force from abutments. As laboratory test indicated, silty clay selected for the dam material is non dispersive soil. But seepage emerged near the conduit results from lack of silty clay compaction.

a) Seepage under Diversion Conduit b) Seepage from Abutments

Figure4. 9: Seepage Problem

c) Reservoir before dam construction d) Reservoir after coffer dam constructed

Figure4. 10: Reservoir Before and After Cofferdam Construction

The SEEP/W program helps to analyze the various pressure conditions, flow conditions and changes in the material properties at any point of the embankment. The pressure condition could be analyzed in different forms such as total pressure, pressure head, pore-water pressure and the hydraulic gradient separately.

4.3.2 Cofferdam Slope Stability Analysis

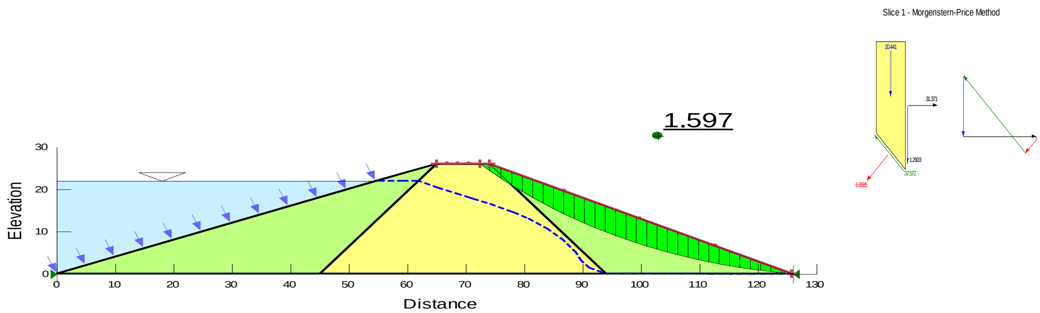
4.3.2.1 Downstream Cofferdam Stability Analysis

Stability analysis has been done with Mohr-coulomb method and the strength parameters defined as follows:-

For core material (Silty clay): unit weight=18.4KN/m^3, cohesion=30kpa, phi=26^0

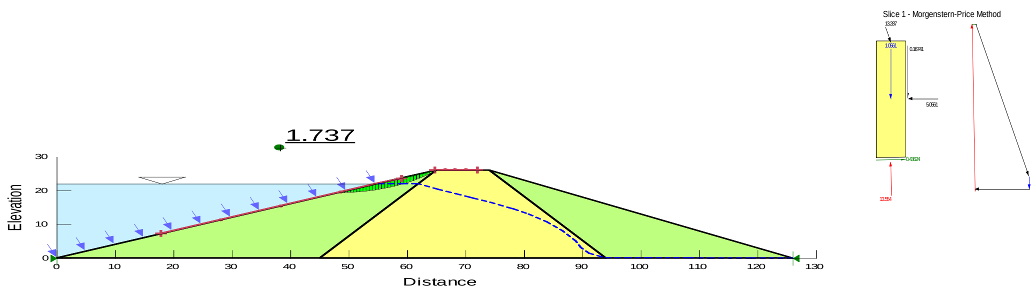
For rock fill: unit weight=20.5KN/m^3, cohesion=0.5kpa, phi=35^0

Factor of safety is calculated using Morgenstern-price method.



a) Constructed D/S slope stability with free body diagram and force polygon using Morgenstern-price method

4.3.2.2 Upstream Cofferdam Stability Analysis



b) U/S slope stability with free body diagram and force polygon using Morgenstern-price method

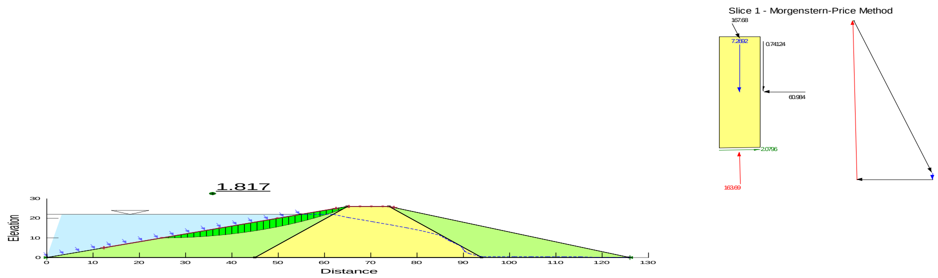
Figure4. 11: Steady-State Analysis f Constructed Cofferdam

Table4. 5: U/S and D/S cofferdam factor of safety

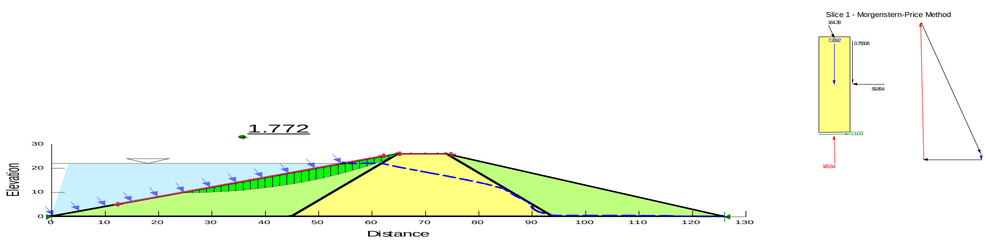
Condition	U/S	D/S	Method
Steady-state	Factor of safety obtained		
	1.737	1.597	Morgenstern-price method

The stability analysis of both U/S and D/S slopes of the cofferdam have been done with the properties of shell material and clay material. The factors of safety for upstream 1.737 and factor of safety for downstream slope is (1.597) so, the slope has enough stability during steady-state. (FOS ≥1.5)

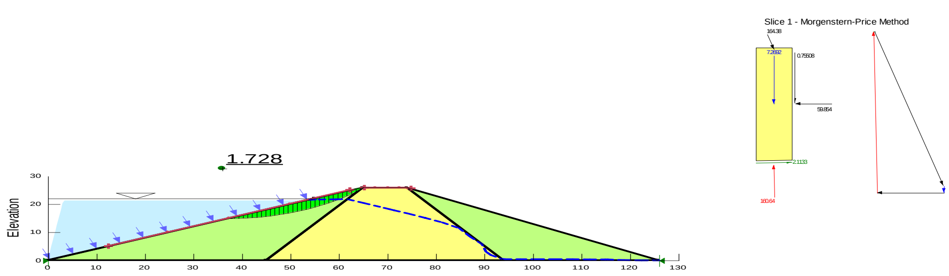
4.3.2.3 Constructed Cofferdam Drawdown Slope Stability Analysis



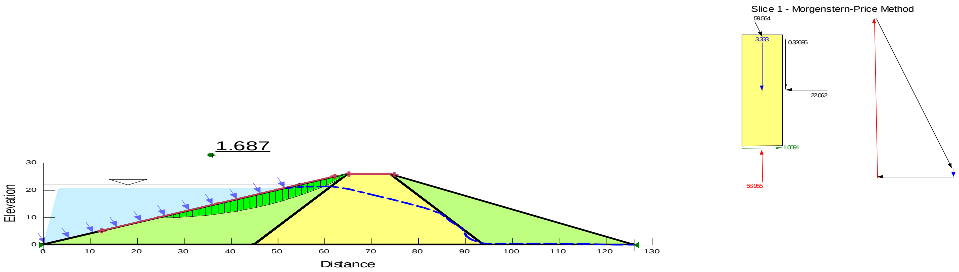
a) Designed cofferdam stability analysis after4.8 hr with free body diagram and force polygon



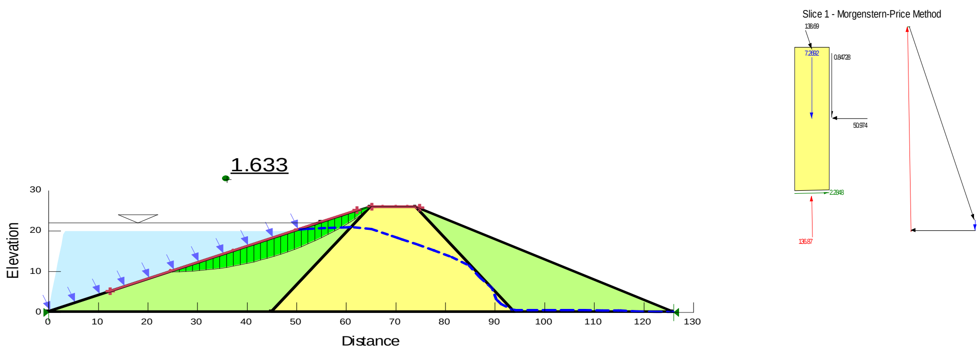
b) Designed cofferdam stability analysis after 12.3hr with free body diagram and force polygon



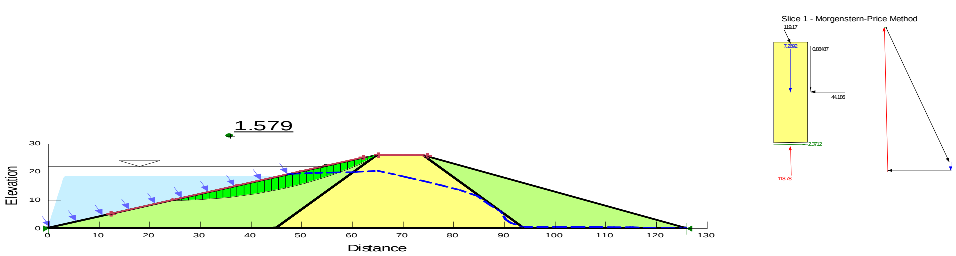
c) Constructed cofferdam stability analysis after 24 hrs with free body diagram and force polygon



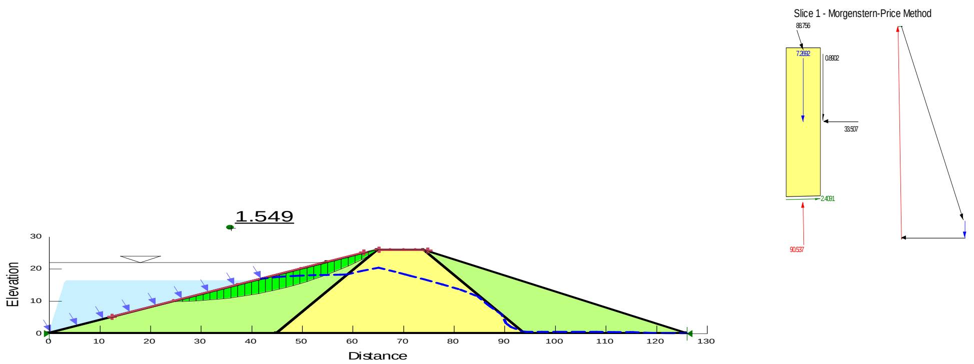
d) Constructed cofferdam stability analysis after 1.76 days with free body diagram and force polygon



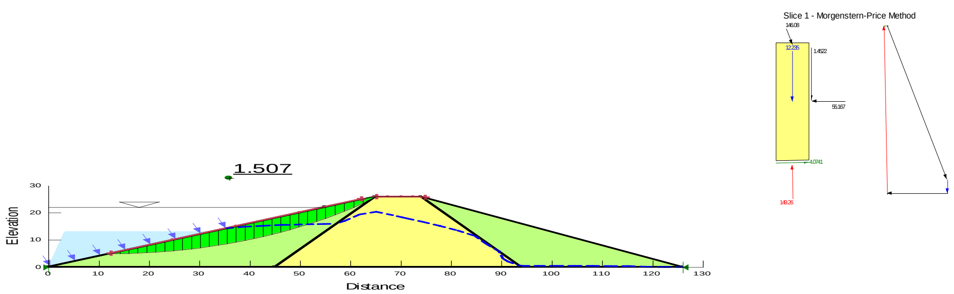
e) Constructed cofferdam stability analysis after 2.94 days with free body diagram and force polygon.



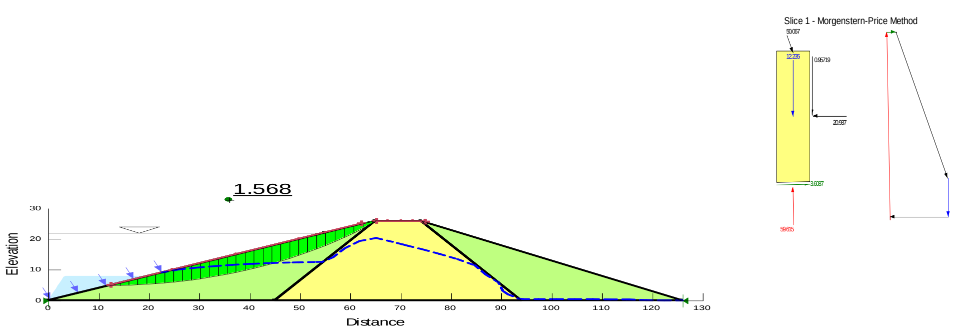
f) Constructed cofferdam stability analysis after 4.78 days with free body diagram and force polygon.



g) Constructed cofferdam stability analysis after 7.65 days with free body diagram and force polygon

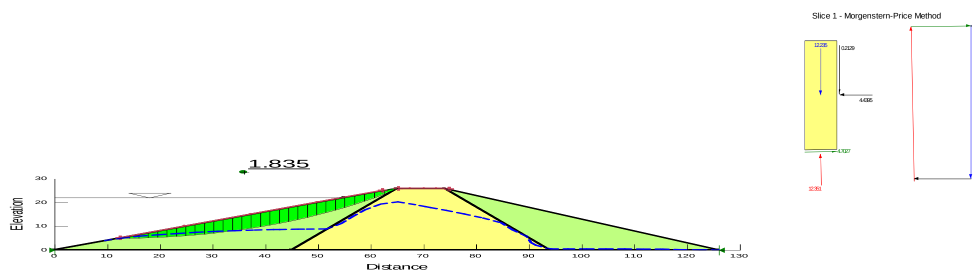


h) Constructed cofferdam stability analysis after 12.1 days with free body diagram and force polygon



i) Constructed cofferdam stability analysis after 19.1 days with free body diagram and force polygon

-



j) Constructed cofferdam stability analysis after 30 days with free body diagram and force polygon

Figure4. 12: transient analysis of u/s side free body diagram and force polygon Morgenstern-price method

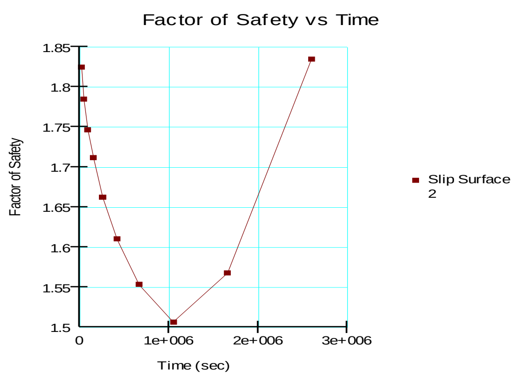


Figure4. 13 factor of safety vs time

Table4. 6: Cofferdam drawdown stability analysis

Time	Safety factor	Method
4.8 hr	1.817	Morgenstern-price method
12.3 hr	1.772	Morgenstern-price method
24 hr	1.728	Morgenstern-price method
1.76 day	1.687	Morgenstern-price method

2.94 days	1.633	Morgenstern-price method
4.78 days	1.579	Morgenstern-price method
7.65 days	1.549	Morgenstern-price method
12.1 days	1.507	Morgenstern-price method
19.1 days	1.568	Morgenstern-price method
30 days	1.835	Morgenstern-price method

The factor of safety is decreases and then increasing until the end of the analysis. The results show that the slope is potentially stable throughout the drawdown. It is because of the obtained value meet the minimum required. (FOS ≥1.3)

4.4 Main Dam Seepage and Slope Stability Analysis

Arjo Dhidhessa embankment dam have been used the same materials and soft ware with the cofferdam.

4.4.1 Steady-State Seepage Analysis

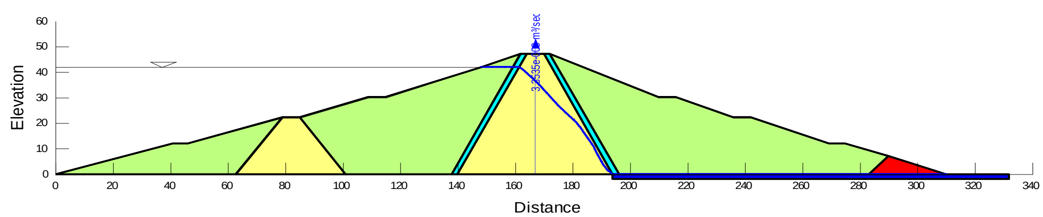
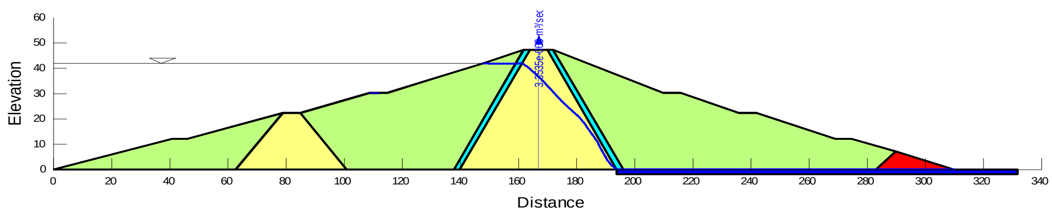


Figure4. 14: steady-state seepage analysis

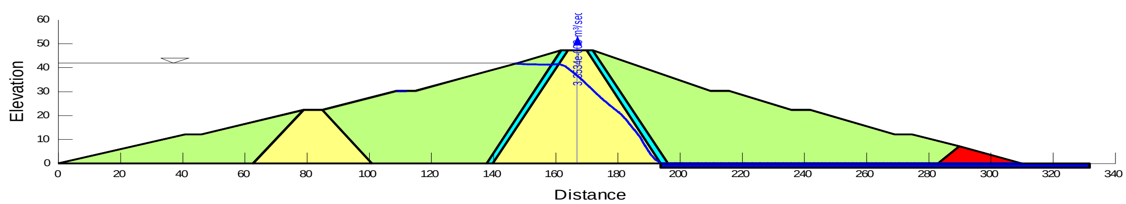
4.4.2 Transient Analysis

During the drawdown the water level of the main dam reduced from 41.6m to 30m. But, the figure below shows only the full supply level. The transient analysis could be done based on the steady-state analysis as parental analysis. Therefore the pressure head and the pore water pressure which obtained from the steady-state analysis are transferred to the transient analysis as the boundary condition. The properties of soil such as permeability and the volumetric water content which defined in steady-state analysis also imported to the transient analysis.

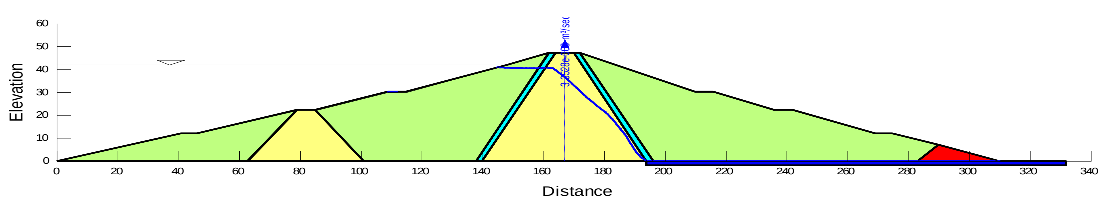
Initially the time duration for analysis was defined as 90days and with 25 time steps and the time increment was exponential manner. Every time steps in the model were saved and only step 1, 5,10,15,20 and 25 figures are shown below.



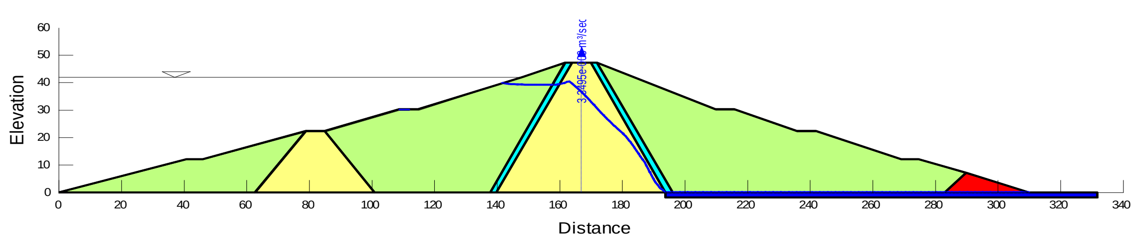
a) Main dam drawdown seepage after 8.64 hr



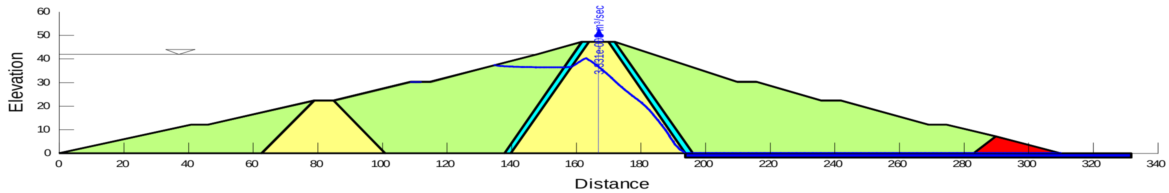
b) Main dam drawdown seepage after 2.48days



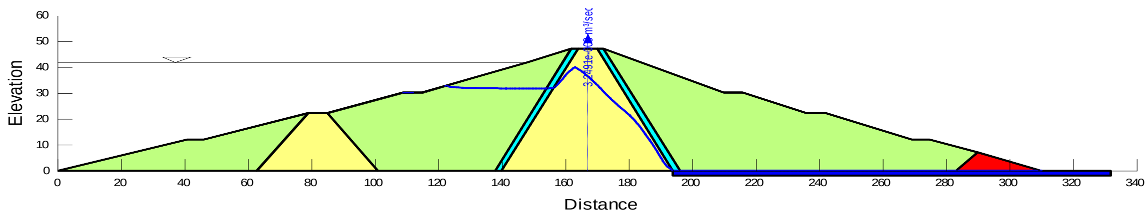
c) Main dam drawdown seepage after 7.68 days



d) Main dam drawdown seepage after 18.6 days.



e) Main dam drawdown seepage after 41.6 days



f) Main dam drawdown seepage after 90 days

Figure4. 15: transient seepage analysis of m

Table4. 7: Main dam total fluxes seepage transient analysis

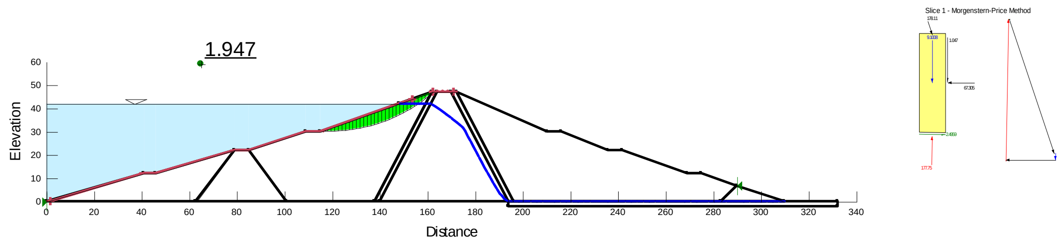
Time	Seepage through the dam ($10^{-8} m^3/s$)
8.64 hr	3.3535
18.7 hr	3.3535
1.26 days	3.3535
1.82 days	3.3534
2.48 days	3.3534
3.23 days	3.3533
4.11 days	3.3533
5.13 days	3.3531
6.31 days	3.3530

7.68 days	3.3528
9.27 days	3.3525
11.1 days	3.3521
13.3 days	3.3515
15.7 days	3.3507
18.6 days	3.3495
22 days	3.3479
25.9 days	3.3455
30.4 days	3.3421
35.6 days	3.3374
41.6 days	3.3310
48.7 days	3.3222
56.8 days	3.3105
66.3 days	3.2949
77.3 days	3.2748
90 days	3.2491

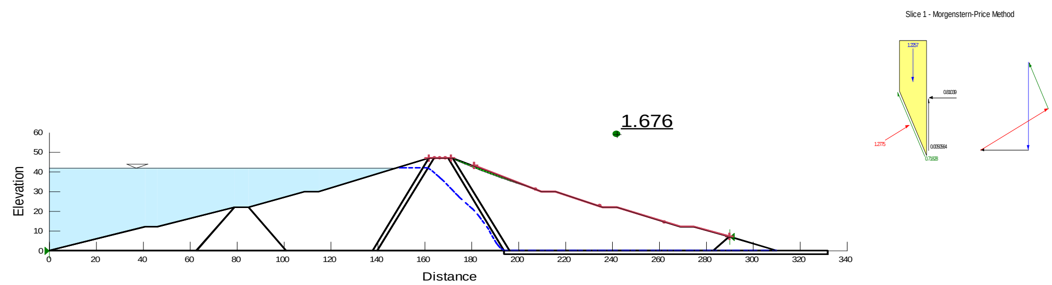
The seepage (total flux) through the embankment continuously reducing with increasing time and is low when compared with the flux through steady analysis. It shows that the lowering pore water pressure doesn't affect the seepage condition.

4.4.3 Main Dam Slope Stability Analysis

4.4.3.1 Steady-state Conditions Upstream Side



a) U/S main dam slope stability analysis.



b) D/S main dam slope stability analysis

Figure4. 16: Steady- state slope stability of main dam

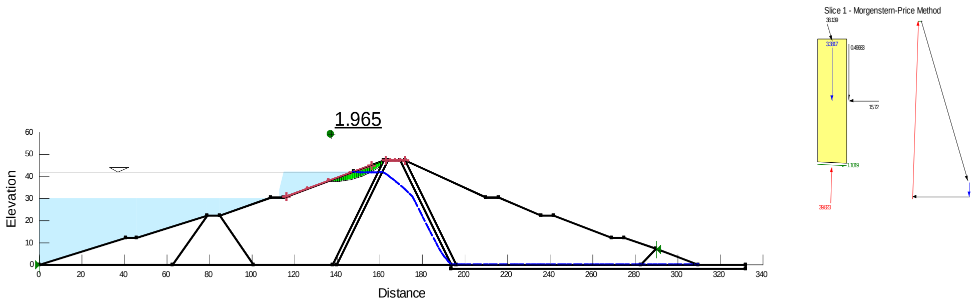
Table4. 8: Main dam U/S and D/S factor of safety

Condition	U/S	D/S
	Factor of safety obtained	
Steady-state	1.947	1.676

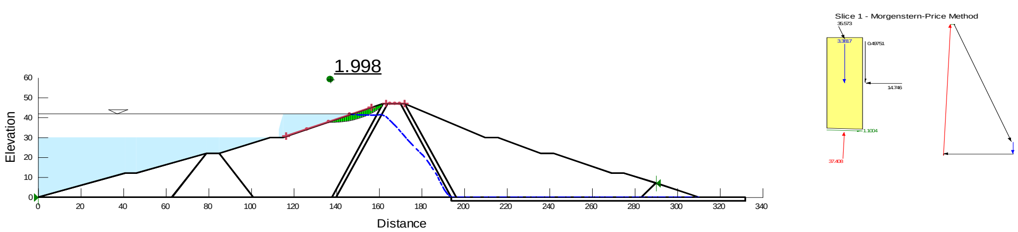
The factor of safety values obtained from the stability analysis of steady anaysis shows that the slope is extremely stable at all. (FOS ≥1.5)

4.4.3.2 Main Dam Transient Analysis Conditions

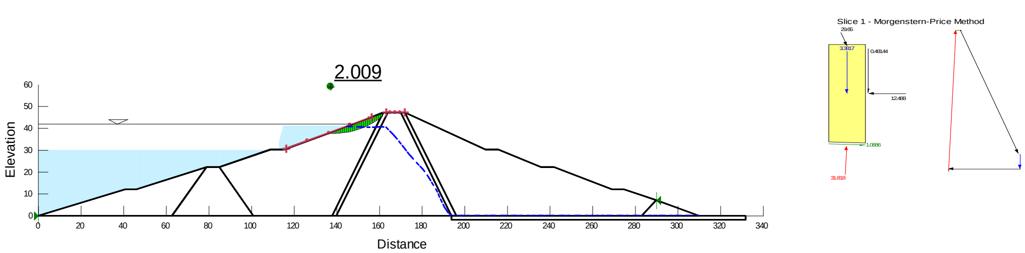
During the drawdown the water level of the main dam reduced from 42m to 30m. But, the figure below shows only the full supply level. The transient analysis could be done based on the steady-state analysis as parental analysis. Therefore the pressure head and the pore water pressure which obtained from the steady-state analysis are transferred to the transient analysis as the boundary condition. The soil strength parameters which defined in steady-state analysis also imported to the transient analysis. Initially the time duration for analysis was defined as 90 days 25 time steps and the time increment was exponential manner. Every time steps in the model were saved and only step 1, 5, 10, 15, 20 and 25 figures are shown below.



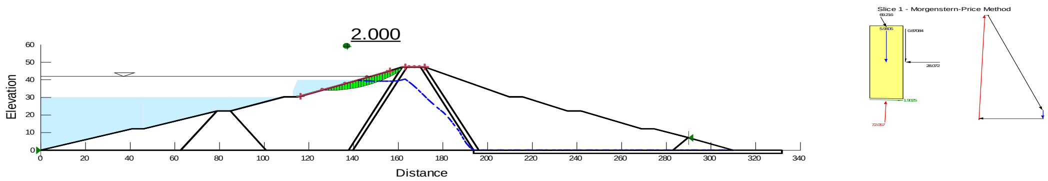
a) Main dam stability analysis after 8.64 hr with free body diagram and force polygon



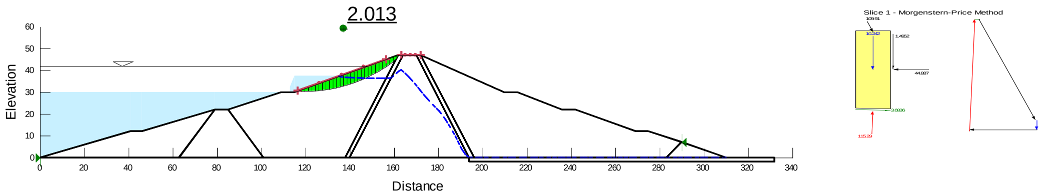
b) Main dam stability analysis after 2.48 days with free body diagram and force polygon



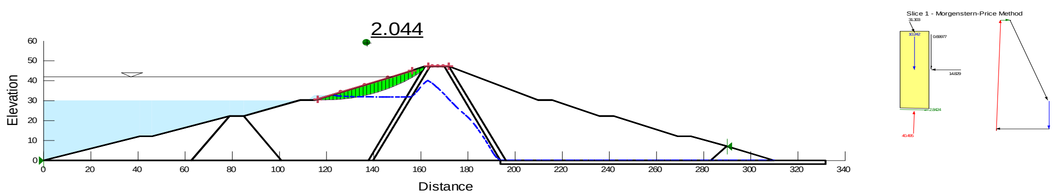
c) Main dam stability analysis after 7.68 days with free body diagram and force polygon



d) Main dam stability analysis after 18.6days with free body diagram and force polygon



e) Main dam stability analysis after 41.6 days with free body diagram and force polygon



f) Main Dam Stability Analysis after 90 days with free body diagram and force polygon

Figure4. 17: Transient analysis with free body diagram and force polygon

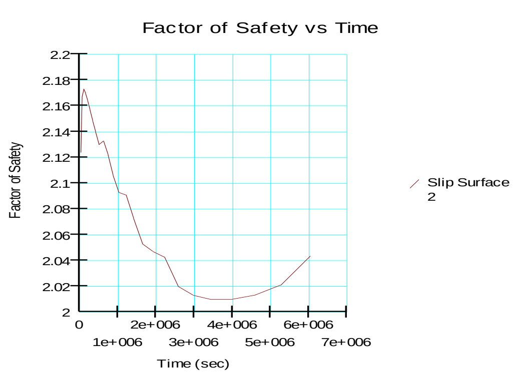


Figure4. 18: factor of safety vs time

Table4. 9: Main dam drawdown slope stability analysis

Time	Factor of safety	Method
8.64 hr	1.965	Morgenstern-price method
18.7 hr	1.996	Morgenstern-price method
1.26 days	2.002	Morgenstern-price method
1.82 days	2.000	Morgenstern-price method
2.48 days	1.998	Morgenstern-price method
3.23 days	1.995	Morgenstern-price method
4.11 days	1.993	Morgenstern-price method
5.13 days	1.994	Morgenstern-price method
6.31 days	1.991	Morgenstern-price method
7.68 days	2.009	Morgenstern-price method
9.27 days	2.012	Morgenstern-price method
11.1 days	2.009	Morgenstern-price method
13.3 days	2.003	Morgenstern-price method
15.7 days	2.010	Morgenstern-price method
18.6 days	2.000	Morgenstern-price method
22 days	1.991	Morgenstern-price method
25.9 days	2.000	Morgenstern-price method
30.4 days	2.012	Morgenstern-price method

35.6 days	2.007	Morgenstern-price method
41.6 days	2.013	Morgenstern-price method
48.7 days	2.010	Morgenstern-price method
56.8 days	2.010	Morgenstern-price method
66.3 days	2.013	Morgenstern-price method
77.3 days	2.021	Morgenstern-price method
90 days	2.044	Morgenstern-price method

The factor of safety is increasing until the end of the analysis. The results show that the slope is potentially stable throughout the drawdown. It is because of the obtained value meet the minimum required. (FOS $\geq$1.3)

5 Conclusion and Recommendation

5.1 Conclusion

- 6m height of Arjo Dhidhessa Cofferdam added without as per design to mitigate flood occurred in July 2015 after the Designed cofferdam construction was completed.
- The increased height of the cofferdam has no significant negative effect on stability of the main dam because of it will be submerged by dead storage in future.
- Seepage not analyzed for Arjo Dhidhessa Coffer dam and main dam. But in this thesis seepage analyzed for Designed Cofferdam, Constructed Cofferdam and Main dam.
- The seepage and stability analysis has been done using the professional version of the popular geotechnical software Geo studio.
- Two fundamental types of finite element seepage analyses: steady-state and transient were analyzed using geo studio software. Results were shown for Arjo Dhidhessa main and cofferdam.
- The total flux discharges through the cofferdam continuously reducing with increasing time.
- The slope stability analysis for cofferdam result shows that the slope is potentially stable throughout steady-state and transient analysis.
- Excessive seepage occurred emerged from the abutments and near the conduit of outlet.
- Water entered in to the abutments during overtopping occurred and seeps in to the core foundation.
- Non dispersive silty clay soil used near concrete structure. However excessive seepage that has been come out near concrete tunnel resulted from lack of proper compaction and cut off wall.
- The total seepage through the Arjo Dhidhessa Embankment dam continuously reducing with increasing time and is low when compared with the flux through

steady analysis. It shows that the lowering pore water pressure doesn’t affect the seepage condition.

- Factor of safety increases as flux discharge decreases and beyond the minimum requirement (1.3) which indicates the dam is extremely stable through transient analysis.

5.2 Recommendation

- Foundation and abutments should be grouted effectively to control excessive seepage occurred.
- Non dispersive soil is recommended near the concrete tunnel at the main dam core foundation and cut off wall to control seepage.
- Arjo Dhidhessa cofferdam construction completed without taken disturbed hydraulic conductivity on site. For main dam it is strictly recommended that disturbed hydraulic conductivity test is mandatory.
- Dam material properties like hydraulic conductivity of filter, horizontal drain, rock toe and toe drain should be tested for the construction of main dam.
- Since strength parameters were tested for shell material and impervious core only in Design Document. But strength parameters of others also required.
- Arjo Dhidhessa Design Document should be reviewed before proceed to main dam construction.

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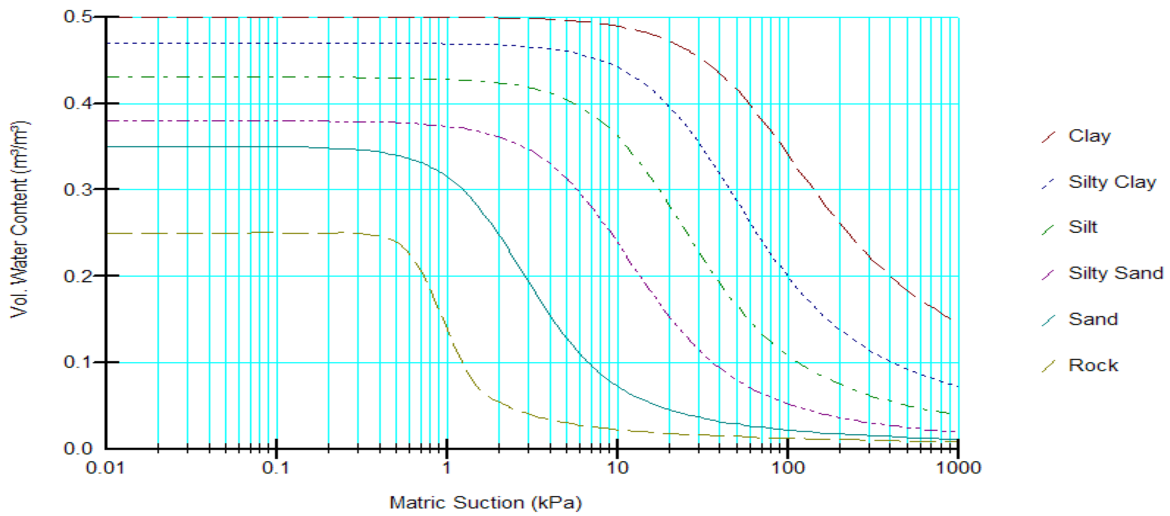
Appendices

Appendix A: sample function



Appendix B: Initial condition steady-state (2)

Report generated using GeoStudio 2007, version 7.10. Copyright © 1991-2008 GEO-SLOPE International Ltd.

File Information

Revision Number: 60

Date: 12/25/2016

Time: 7:24:13 AM

File Name: modern drawing - Copy (2) - Copy.gsz

Directory: C:\Users\lapowner\Desktop\geo stu filter\

Project Settings

Length (L) Units: meters

Time (t) Units: Seconds

Force (F) Units: kN

Pressure (p) Units: kPa

Mass (M) Units: g

Mass Flux Units: g/sec

Unit Weight of Water: 9.807 kN/m³

View: 2D

Analysis Settings

Initial condition steady-state (2)

Description: full reservoir condition

Kind: SEEP/W

Method: Steady-State

Settings

Include Air Flow: No

Control

Apply Runoff: Yes

Convergence

Maximum Number of Iterations: 50

Tolerance: 0.1

Maximum Change in K: 1

Rate of Change in K: 1.1

Minimum Change in K: 0.0001

Equation Solver: Parallel Direct

Potential Seepage Max # of Reviews: 10

Time

Starting Time: 0 sec

Duration: 0 sec

Ending Time: 0 sec

Materials

silty clay

Model: Saturated / Unsaturated

Hydraulic

K-Function: silty clay conductivity

Vol. WC. Function: silty clay

K-Ratio: 1

K-Direction: 0 °

sand

Model: Saturated / Unsaturated

Hydraulic

K-Function: sand conductivity

Vol. WC. Function: sand

K-Ratio: 1

K-Direction: 0 °

Silt filter

Model: Saturated / Unsaturated

Hydraulic

K-Function: filter conductivity

Vol. WC. Function: silt filter

K-Ratio: 1

K-Direction: 0 °

horizontal blanket

Model: Saturated Only

Hydraulic

K-Sat: 0.189 m/sec

Volumetric Water Content: 0 m^3/m^3

Mv: 0 /kPa

K-Ratio: 1

K-Direction: 0 °

rock toe

Model: Saturated Only

Hydraulic

K-Sat: 1 m/sec

Volumetric Water Content: 0 m³/m³

Mv: 0 /kPa

K-Ratio: 1

K-Direction: 0 °

Boundary Conditions

Zero Pressure

Type: Pressure Head 0

Potential Seepage Face

Review: true

Type: Total Flux (Q) 0

resevoir level=42

Type: Head (H) 42

Flux Sections

Flux Section 1

Coordinates

Coordinate: (167, 0) m

Coordinate: (167, 53) m

K Functions

silty clay conductivity

Model: Data Point Function

Function: X-Conductivity vs. Pore-Water Pressure

Curve Fit to Data: 100 %

Segment Curvature: 100 %

K-Saturation: 1.19e-009

Data Points: Matric Suction (kPa), X-Conductivity (m/sec)

Data Point: (0.01, 1.19e-009)

Data Point: (0.018329807, 1.1871315e-009)

Data Point: (0.033598183, 1.182522e-009)

Data Point: (0.061584821, 1.1751441e-009)

Data Point: (0.11288379, 1.163344e-009)

Data Point: (0.20691381, 1.1445112e-009)

Data Point: (0.37926902, 1.1145722e-009)

Data Point: (0.6951928, 1.0672797e-009)

Data Point: (1.274275, 9.9342048e-010)

Data Point: (2.3357215, 8.805569e-010)

Data Point: (4.2813324, 7.1547212e-010)

Data Point: (7.8475997, 4.959336e-010)

Data Point: (14.384499, 2.5820011e-010)

Data Point: (26.366509, 8.5402107e-011)

Data Point: (48.329302, 1.658883e-011)

Data Point: (88.586679, 2.1411197e-012)

Data Point: (162.37767, 2.2333306e-013)

Data Point: (297.63514, 2.1356473e-014)

Data Point: (545.55948, 1.9786707e-015)

Data Point: (1000, 1.8131841e-016)

Estimation Properties

Volume Water Content Function: silty clay

Hydraulic K Sat: 1.19e-009 m/sec

Hyd. K-Function Estimation Method: Van Genuchten Function

Maximum: 1000

Minimum: 0.01

Num. Points: 20

Residual Water Content: 0.047 m^3/m^3

sand conductivity

Model: Data Point Function

Function: X-Conductivity vs. Pore-Water Pressure

Curve Fit to Data: 100 %

Segment Curvature: 100 %

K-Saturation: 0.0053

Data Points: Matric Suction (kPa), X-Conductivity (m/sec)

Data Point: (0.01, 0.0053)

Data Point: (0.018329807, 0.0052847576)

Data Point: (0.033598183, 0.0052519784)

Data Point: (0.061584821, 0.0051818558)

Data Point: (0.11288379, 0.0050324609)

Data Point: (0.20691381, 0.0047178697)

Data Point: (0.37926902, 0.0040790817)

Data Point: (0.6951928, 0.0029126101)

Data Point: (1.274275, 0.0013400312)

Data Point: (2.3357215, 0.00027709196)

Data Point: (4.2813324, 2.4485307e-005)

Data Point: (7.8475997, 1.3504128e-006)

Data Point: (14.384499, 6.3313953e-008)

Data Point: (26.366509, 2.8372619e-009)

Data Point: (48.329302, 1.2565675e-010)

Data Point: (88.586679, 5.5483179e-012)

Data Point: (162.37767, 2.4479588e-013)

Data Point: (297.63514, 1.0798465e-014)

Data Point: (545.55948, 4.7631954e-016)

Data Point: (1000, 2.1010157e-017)

Estimation Properties

Volume Water Content Function: sand

Hydraulic K Sat: 0.0053 m/sec

Hyd. K-Function Estimation Method: Van Genuchten Function

Maximum: 1000

Minimum: 0.01

Num. Points: 20

Residual Water Content: 0.035 m^3/m^3

Filter conductivity

Model: Data Point Function

Function: X-Conductivity vs. Pore-Water Pressure

Curve Fit to Data: 100 %

Segment Curvature: 100 %

K-Saturation: 3.5e-006

Data Points: Matric Suction (kPa), X-Conductivity (m/sec)

Data Point: (0.01, 3.5e-006)

Data Point: (0.018329807, 3.4929796e-006)

Data Point: (0.033598183, 3.4808315e-006)

Data Point: (0.061584821, 3.4598971e-006)

Data Point: (0.11288379, 3.4238524e-006)

Data Point: (0.20691381, 3.3619391e-006)

Data Point: (0.37926902, 3.2561304e-006)

Data Point: (0.6951928, 3.0769564e-006)

Data Point: (1.274275, 2.7792167e-006)

Data Point: (2.3357215, 2.3047735e-006)

Data Point: (4.2813324, 1.6184816e-006)

Data Point: (7.8475997, 8.265175e-007)

Data Point: (14.384499, 2.4907294e-007)

Data Point: (26.366509, 4.0622201e-008)

Data Point: (48.329302, 4.2585147e-009)

Data Point: (88.586679, 3.6224192e-010)

Data Point: (162.37767, 2.852358e-011)

Data Point: (297.63514, 2.1888522e-012)

Data Point: (545.55948, 1.6658399e-013)

Data Point: (1000, 1.2644731e-014)

Estimation Properties

Volume Water Content Function: silt filter

Hydraulic K Sat: 3.5e-006 m/sec

Hyd. K-Function Estimation Method: Van Genuchten Function

Maximum: 1000

Minimum: 0.01

Num. Points: 20

Residual Water Content: 0.043 m^3/m^3

Vol. Water Content Functions

silty clay

Model: Data Point Function

Function: Vol. Water Content vs. Pore-Water Pressure

Curve Fit to Data: 100 %

Segment Curvature: 100 %

Mv: 0.0001 /kPa

Saturated Water Content: 0 m^3/m^3

Porosity: 0.47000014

Data Points: Matric Suction (kPa), Vol. Water Content (m^3/m^3)

Data Point: (0.01, 0.46999921)

Data Point: (0.018329807, 0.46999814)

Data Point: (0.033598183, 0.46999551)

Data Point: (0.061584821, 0.46998893)

Data Point: (0.11288379, 0.46997231)

Data Point: (0.20691381, 0.46992983)

Data Point: (0.37926902, 0.46982056)

Data Point: (0.6951928, 0.46953817)

Data Point: (1.274275, 0.46880701)

Data Point: (2.3357215, 0.46691806)

Data Point: (4.2813324, 0.46208988)

Data Point: (7.8475997, 0.4501215)

Data Point: (14.384499, 0.42263251)

Data Point: (26.366509, 0.36916531)

Data Point: (48.329302, 0.29141957)

Data Point: (88.586679, 0.21290397)

Data Point: (162.37767, 0.15384611)

Data Point: (297.63514, 0.11491867)

Data Point: (545.55948, 0.089330281)

Data Point: (1000, 0.071542774)

Estimation Properties

Vol. WC Estimation Method: Sample functions

Sample Material: Silty Clay

Saturated Water Content: 0.47 m³/m³

Liquid Limit: 0 %

Diameter at 10% passing: 0

Diameter at 60% passing: 0

Maximum: 1000

Minimum: 0.01

Num. Points: 20

Sand

Model: Data Point Function

Function: Vol. Water Content vs. Pore-Water Pressure

Curve Fit to Data: 100 %

Segment Curvature: 100 %

Saturated Water Content: 0 m³/m³

Porosity: 0.35056974

Data Points: Matric Suction (kPa), Vol. Water Content (m³/m³)

Data Point: (0.01, 0.34999498)

Data Point: (0.018329807, 0.34998383)

Data Point: (0.033598183, 0.34994752)

Data Point: (0.061584821, 0.34982895)

Data Point: (0.11288379, 0.34944161)

Data Point: (0.20691381, 0.3481802)

Data Point: (0.37926902, 0.34412361)

Data Point: (0.6951928, 0.33160433)

Data Point: (1.274275, 0.2974227)

Data Point: (2.3357215, 0.2284591)

Data Point: (4.2813324, 0.14584461)

Data Point: (7.8475997, 0.08760768)

Data Point: (14.384499, 0.056036002)

Data Point: (26.366509, 0.039058567)

Data Point: (48.329302, 0.029143699)

Data Point: (88.586679, 0.022827024)

Data Point: (162.37767, 0.018499961)

Data Point: (297.63514, 0.015352004)

Data Point: (545.55948, 0.012935626)

Data Point: (1000, 0.010982802)

Estimation Properties

Vol. WC Estimation Method: Sample functions

Sample Material: Sand

Saturated Water Content: 0.35 m^3/m^3

Liquid Limit: 0 %

Diameter at 10% passing: 0

Diameter at 60% passing: 0

Maximum: 1000

Minimum: 0.01

Num. Points: 20

Silt filter

Model: Data Point Function

Function: Vol. Water Content vs. Pore-Water Pressure

Curve Fit to Data: 100 %

Segment Curvature: 100 %

Mv: 0 /kPa

Saturated Water Content: 0 m^3/m^3

Porosity: 0.42999857

Data Points: Matric Suction (kPa), Vol. Water Content (m^3/m^3)

Data Point: (0.01, 0.42999857)

Data Point: (0.018329807, 0.4299964)

Data Point: (0.033598183, 0.42999077)

Data Point: (0.061584821, 0.42997606)

Data Point: (0.11288379, 0.42993738)

Data Point: (0.20691381, 0.42983525)

Data Point: (0.37926902, 0.42956493)

Data Point: (0.6951928, 0.42884918)

Data Point: (1.274275, 0.42696041)

Data Point: (2.3357215, 0.42203526)

Data Point: (4.2813324, 0.40960662)

Data Point: (7.8475997, 0.38070742)

Data Point: (14.384499, 0.3245819)

Data Point: (26.366509, 0.24512312)

Data Point: (48.329302, 0.16913453)

Data Point: (88.586679, 0.1157285)

Data Point: (162.37767, 0.082719569)

Data Point: (297.63514, 0.062216403)

Data Point: (545.55948, 0.048701844)

Data Point: (1000, 0.039127909)

Estimation Properties

Vol. WC Estimation Method: Sample functions

Sample Material: Silt

Saturated Water Content: 0.43 m³/m³

Liquid Limit: 0 %

Diameter at 10% passing: 0

Diameter at 60% passing: 0

Maximum: 1000

Minimum: 0.01

Num. Points: 20

Regions

	Material	Points	Area (m²)
Region 1	sand	1,2,3,4,5	691
Region 2	silty clay	4,6,7,5	484
Region 3	sand	6,8,9,30,10,11,7	1873.5
Region 4	silt filter	10,12,13,11	94
Region 5	silty clay	12,14,15,13	1410
Region 6	silt filter	14,16,17,15	94
Region 7	sand	16,18,19,20,21,22,23,24,25,17	2380
Region 8	rock toe	24,26,25	94.5
Region 9	horizontal blanket	26,27,28,29,15,17,25	276

Lines

	Start Point	End Point	Hydraulic Boundary
Line 1	1	2	resevoir level=42

Line 2	2	3	resevoir level=42
Line 3	3	4	resevoir level=42
Line 4	4	5	
Line 5	5	1	
Line 6	4	6	resevoir level=42
Line 7	6	7	
Line 8	7	5	
Line 9	6	8	resevoir level=42
Line 10	8	9	resevoir level=42
Line 11	10	11	
Line 12	11	7	
Line 13	10	12	
Line 14	12	13	
Line 15	13	11	
Line 16	12	14	
Line 17	14	15	
Line 18	15	13	
Line 19	14	16	
Line 20	16	17	
Line 21	17	15	
Line 22	16	18	Potential Seepage Face
Line 23	18	19	Potential Seepage Face
Line 24	19	20	Potential Seepage Face

Line 25	20	21	Potential Seepage Face
Line 26	21	22	Potential Seepage Face
Line 27	22	23	Potential Seepage Face
Line 28	23	24	
Line 29	24	25	
Line 30	25	17	
Line 31	24	26	Potential Seepage Face
Line 32	26	25	
Line 33	26	27	
Line 34	27	28	
Line 35	28	29	
Line 36	29	15	
Line 37	9	30	resevoir level=42
Line 38	30	10	

Points

	X (m)	Y (m)	Hydraulic Boundary
Point 1	0	0	
Point 2	41	12	
Point 3	46	12	
Point 4	79	22	
Point 5	63	0	
Point 6	85	22	
Point 7	101	0	
Point 8	109	30	

Point 9	115	30	
Point 10	162	47	
Point 11	138	0	
Point 12	164	47	
Point 13	140	0	
Point 14	170	47	
Point 15	194	0	
Point 16	172	47	
Point 17	196	0	
Point 18	210	30	
Point 19	216	30	
Point 20	236	22	
Point 21	242	22	
Point 22	269	12	
Point 23	275	12	
Point 24	290	7	
Point 25	283	0	
Point 26	310	0	Zero Pressure
Point 27	332	0	
Point 28	332	-2	
Point 29	194	-2	
Point 30	148.17647	42	

Appendix B: Transient Seepage

Report generated using GeoStudio 2007, version 7.10. Copyright © 1991-2008 GEO-SLOPE International Ltd.

Of Steps: 25

Step Generation Method: Exponential

Initial Increment Size: 31104 sec

Save Steps Every: 1

Use Adaptive Time Stepping: No

Regions

	Material	Points	Area (m^2)
Region 1	sand	1,2,3,4,5	691
Region 2	silty clay	4,6,7,5	484
Region 3	sand	6,8,9,30,10,11,7	1873.5
Region 4	silt filter	10,12,13,11	94
Region 5	silty clay	12,14,15,13	1410
Region 6	silt filter	14,16,17,15	94
Region 7	sand	16,18,19,20,21,22,23,24,25,17	2380
Region 8	rock toe	24,26,25	94.5
Region 9	horizontal blanket	26,27,28,29,15,17,25	276

Lines

	Start Point	End Point	Hydraulic Boundary
Line 1	1	2	reservoir drawdown
Line 2	2	3	reservoir drawdown
Line 3	3	4	reservoir drawdown
Line 4	4	5	

Line 5	5	1	
Line 6	4	6	reservoir drawdown
Line 7	6	7	
Line 8	7	5	
Line 9	6	8	reservoir drawdown
Line 10	8	9	reservoir drawdown
Line 11	10	11	
Line 12	11	7	
Line 13	10	12	
Line 14	12	13	
Line 15	13	11	
Line 16	12	14	
Line 17	14	15	
Line 18	15	13	
Line 19	14	16	
Line 20	16	17	
Line 21	17	15	
Line 22	16	18	Potential Seepage Face
Line 23	18	19	Potential Seepage Face
Line 24	19	20	Potential Seepage Face
Line 25	20	21	Potential Seepage Face
Line 26	21	22	Potential Seepage Face
Line 27	22	23	Potential Seepage Face

Line 28	23	24	
Line 29	24	25	
Line 30	25	17	
Line 31	24	26	Potential Seepage Face
Line 32	26	25	
Line 33	26	27	
Line 34	27	28	
Line 35	28	29	
Line 36	29	15	
Line 37	9	30	seepage face
Line 38	30	10	

Points

	X (m)	Y (m)	Hydraulic Boundary
Point 1	0	0	
Point 2	41	12	
Point 3	46	12	
Point 4	79	22	
Point 5	63	0	
Point 6	85	22	
Point 7	101	0	
Point 8	109	30	
Point 9	115	30	
Point 10	162	47	
Point 11	138	0	

Point 12	164	47	
Point 13	140	0	
Point 14	170	47	
Point 15	194	0	
Point 16	172	47	
Point 17	196	0	
Point 18	210	30	
Point 19	216	30	
Point 20	236	22	
Point 21	242	22	
Point 22	269	12	
Point 23	275	12	
Point 24	290	7	
Point 25	283	0	
Point 26	310	0	Zero Pressure
Point 27	332	0	
Point 28	332	-2	
Point 29	194	-2	
Point 30	148.17647	42	

Appendix D: U/S steady state slope stability

Slope Stability(d/s)

File Information

Revision Number: 60

Date: 12/25/2016

Time: 7:24:13 AM

File Name: modern drawing - Copy (2) - Copy.gsz

Directory: C:\Users\lapowner\Desktop\geo stu filter\

Project Settings

Length(L) Units: meters

Time(t) Units: Seconds

Force(F) Units: kN

Pressure(p) Units: kPa

Strength Units: kPa

Unit Weight of Water: 9.807 kN/m³

View: 2D

Analysis Settings

Slope Stability(d/s)

Description: full reservoir condition

Kind: SLOPE/W

Parent: initial condition steady-state (2)

Method: Morgenstern-Price

Settings

Side Function

Interslice force function option: Half-Sine

PWP Conditions Source: Parent Analysis

SlipSurface

Direction of movement: Right to Left

Allow Passive Mode: No

Slip Surface Option: Entry and Exit

Critical slip surfaces saved: 1

Optimize Critical Slip Surface Location: No

Tension Crack

Tension Crack Option: (none)

FOS Distribution

FOS Calculation Option: Constant

Advanced

Number of Slices: 30

Optimization Tolerance: 0.01

Minimum Slip Surface Depth: 0.1 m

Minimum Slice Width: 0.1 m

Optimization Maximum Iterations: 2000

Optimization Convergence Tolerance: 1e-007

Starting Optimization Points: 8

Ending Optimization Points: 16

Complete Passes per Insertion: 1

Materials

silty clay

Model: Mohr-Coulomb

Unit Weight: 18.4 kN/m³

Cohesion: 30 kPa

Phi: 26 °

Phi-B: 0 °

sand

Model: Mohr-Coulomb

Unit Weight: 20.5 kN/m³

Cohesion: 0.5 kPa

Phi: 35 °

Phi-B: 0 °

Slip Surface Entry and Exit

Left Projection: Range

Left-Zone Left Coordinate: (1.5090307, 0.44166753) m

Left-Zone Right Coordinate: (153.70588, 43.999999) m

Left-Zone Increment: 4

Right Projection: Range

Right-Zone Left Coordinate: (162.12197, 47) m

Right-Zone Right Coordinate: (171, 47) m

Right-Zone Increment: 4

Radius Increments: 4

Slip Surface Limits

Left Coordinate: (0, 0) m

Right Coordinate: (290, 7) m

Regions

	Material	Points	Area (m^2)
Region 1	sand	1,2,3,4,5	691
Region 2	silty clay	4,6,7,5	484
Region 3	sand	6,8,9,30,10,11,7	1873.5
Region 4		10,12,13,11	94
Region 5	silty clay	12,14,15,13	1410
Region 6		14,16,17,15	94
Region 7	sand	16,18,19,20,21,22,23,24,25,17	2380
Region 8		24,26,25	94.5
Region 9		26,27,28,29,15,17,25	276

Points

	X (m)	Y (m)
Point 1	0	0
Point 2	41	12
Point 3	46	12
Point 4	79	22
Point 5	63	0

Point 6	85	22
Point 7	101	0
Point 8	109	30
Point 9	115	30
Point 10	162	47
Point 11	138	0
Point 12	164	47
Point 13	140	0
Point 14	170	47
Point 15	194	0
Point 16	172	47
Point 17	196	0
Point 18	210	30
Point 19	216	30
Point 20	236	22
Point 21	242	22
Point 22	269	12
Point 23	275	12
Point 24	290	7
Point 25	283	0
Point 26	310	0
Point 27	332	0
Point 28	332	-2

Point 29	194	-2
Point 30	148.17647	42

Appendix E: Transient slope stability

Slope Stabilitydrawdown (U/S)

Report generated using GeoStudio 2007, version 7.10. Copyright © 1991-2008 GEO-SLOPE International Ltd.

File Information

Revision Number: 60

Date: 12/25/2016

Time: 7:24:13 AM

File Name: modern drawing - Copy (2) - Copy.gsz

Directory: C:\Users\lapowner\Desktop\geo stu filter\

Project Settings

Length(L) Units: meters

Time(t) Units: Seconds

Force(F) Units: kN

Pressure(p) Units: kPa

Strength Units: kPa

Unit Weight of Water: 9.807 kN/m³

View: 2D

Analysis Settings

Slope Stabilitydrawdown (U/S)

Description: drawdown

Kind: SLOPE/W

Parent: Transient Seepage

Method: Morgenstern-Price

Settings

Side Function

Interslice force function option: Half-Sine

PWP Conditions Source: Parent Analysis

SlipSurface

Direction of movement: Right to Left

Allow Passive Mode: No

Slip Surface Option: Entry and Exit

Critical slip surfaces saved: 1

Optimize Critical Slip Surface Location: No

Tension Crack

Tension Crack Option: (none)

FOS Distribution

FOS Calculation Option: Constant

Advanced

Number of Slices: 30

Optimization Tolerance: 0.01

Minimum Slip Surface Depth: 0.1 m

Minimum Slice Width: 0.1 m

Optimization Maximum Iterations: 2000

Optimization Convergence Tolerance: 1e-007

Starting Optimization Points: 8

Ending Optimization Points: 16

Complete Passes per Insertion: 1

Materials

silty clay

Model: Mohr-Coulomb

Unit Weight: 18.4 kN/m³

Cohesion: 30 kPa

Phi: 26 °

Phi-B: 0 °

sand

Model: Mohr-Coulomb

Unit Weight: 20.5 kN/m³

Cohesion: 0.5 kPa

Phi: 35 °

Phi-B: 0 °

Slip Surface Entry and Exit

Left Projection: Range

Left-Zone Left Coordinate: (116.36051, 30.492099) m

Left-Zone Right Coordinate: (156.47059, 45.000001) m

Left-Zone Increment: 4

Right Projection: Range

Right-Zone Left Coordinate: (163.14646, 47) m

Right-Zone Right Coordinate: (172, 47) m

Right-Zone Increment: 4

Radius Increments: 4

Slip Surface Limits

Left Coordinate: (0, 0) m

Right Coordinate: (290, 7) m

Regions

	Material	Points	Area (m^2)
Region 1	sand	1,2,3,4,5	691
Region 2	silty clay	4,6,7,5	484
Region 3	sand	6,8,9,30,10,11,7	1873.5
Region 4		10,12,13,11	94
Region 5	silty clay	12,14,15,13	1410
Region 6		14,16,17,15	94
Region 7	sand	16,18,19,20,21,22,23,24,25,17	2380
Region 8		24,26,25	94.5
Region 9		26,27,28,29,15,17,25	276

Points

	X (m)	Y (m)
Point 1	0	0
Point 2	41	12
Point 3	46	12
Point 4	79	22
Point 5	63	0
Point 6	85	22
Point 7	101	0
Point 8	109	30
Point 9	115	30
Point 10	162	47
Point 11	138	0
Point 12	164	47
Point 13	140	0
Point 14	170	47
Point 15	194	0
Point 16	172	47
Point 17	196	0
Point 18	210	30
Point 19	216	30
Point 20	236	22
Point 21	242	22

Point 22	269	12
Point 23	275	12
Point 24	290	7
Point 25	283	0
Point 26	310	0
Point 27	332	0
Point 28	332	-2
Point 29	194	-2
Point 30	148.17647	42